DESIGN EXAMPLES

DESIGN EXAMPLES
SECTION 8

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SYMBOLS USED IN THIS SECTION

(Symbols Used are listed separately for each Design Example)

DESIGN EXAMPLES

DISCLAIMER

The information in this manual is provided as a guide to assist you with your design and in writing your own specifications.

Installation conditions, including soil and structure conditions, vary widely from location to location and from point to point on a site.

Independent engineering analysis and consulting state and local building codes and authorities should be conducted prior to any installation to ascertain and verify compliance to relevant rules, regulations and requirements.

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Hubbell Power Systems, Inc., does NOT warrant the work of its dealers/installing contractors in the installation of CHANCE® Civil Construction foundation support products.

DESIGN EXAMPLE 1
ATLAS RESISTANCE® PIERS

SYMBOLS USED IN THIS DESIGN EXAMPLE

SPT	Standard Penetration Test	8-4
N	Standard Penetration Test Blow Count	8-4
P	Total Live Load	8-5
DL	Dead Load	8-5
LL	Live Load	8-5
SL	Snow Load	8-5
W	Soil Load	8-5
S_K	Snow Load Requirement Factor	8-5
P_W	Working Pier Load	8-5
x	Pier Spacing	8-5
FS	Factor of Safety	8-5
FS_h	Factor of Safety for Mechanical Strength of Hardware	8-5
$R_{W\ ULT}$	Ultimate Hardware Strength based on Structural Weight	8-5
$R_{h\ ULT}$	Ultimate Hardware Strength	8-5
x_{max}	Maximum Pier Spacing Based on Hardware Capacity	8-5
FS_p	Proof Load Factor of Safety	8-6
R_p	Installation Force to Achieve Proof Load	8-6
$R_{h\ MAX}$	Maximum Installation Force Based on Ultimate Capacity of Hardware	8-6
$L_{p\ MAX}$	Maximum Free Span Between Piers	8-6

Type of Structure

The structure is a two-story, 20' x 40' frame residence with full brick veneer siding located in the Midwest. The house sits on 8" thick by 8' high cast concrete basement walls with steel reinforced concrete footings 1'-8" wide by 1'-0 thick. The roof is composition shingles over 1/2" plywood decking and felt underlayment. There is six feet of peaty clay soil overburden present.

Preliminary Investigation

Settlement is evident in portions of the structure of 2-1/2". Checking with local building officials reveals no special controlling codes for underpinning existing structures that must be observed. Preliminary geotechnical information indicates the footing is situated in peaty clay type soil with Standard Penetration Test (SPT) "N" values of six and higher. This soil extends to a depth of 15 feet where a dense glacial till exists. It is determined that the glacial till layer will serve as an adequate bearing stratum for the ATLAS RESISTANCE® Piers.

Preliminary Estimate of Total Live Load on Footing

P = Dead Load (DL) + Live Load (LL) + Snow Load (SL) + Soil Load (W) **Equation 8-1**

P = (1,890 + 667 + 120 + 2,310) = 4,987 lb/ft

(See Tables 4-2, 4-4 and 4-5 in Section 4 for DL, LL and W).

where:	DL	=	1,890 lb/ft
	LL	=	667 lb/ft
	SL	=	S_K x [(l x w) / 2 (l + w)] where l and w are the building dimensions
	S_K	=	Snow load requirement factor = 18 lb/ft^2 (for this example)
	SL	=	18 lb/ft2 x (800 / 120) ft = 120 lb/ft
	W	=	W_1 + W_2 = (330 + 1,980) lb/ft = 2,310 lb/ft

ATLAS RESISTANCE® Pier Selection

While the ATLAS RESISTANCE® Continuous Lift Pier could be used for this application, the small lift required makes it unnecessary. The ATLAS RESISTANCE® Predrilled Pier is not a good choice here due to the absence of a hard, impenetrable layer above the intended bearing stratum. Therefore, the ATLAS RESISTANCE® 2-Piece Standard Pier is selected for strength and economy. The more expensive ATLAS RESISTANCE® Plate Pier could also be attached to the concrete basement wall and used for this application. Since there are suitable soils with "N" counts above four, there is no need to sleeve the pier pipe for added stiffness.

Pier Spacing

Using the information obtained about the stem wall and footing to be supported, and applying sound engineering judgment, the nominal pier spacing based on the foundation system's ability to span between piers is estimated at about eight feet. This puts the nominal working pier load (P_W) at:

P_W = (x) x (P) = 8 ft x 4,987 lb/ft = 39,896 lbs **Equation 8-2**

where:	x	=	Selected pier spacing = 8 ft
	P	=	Line load on footing = 4,987 lb/ft

Factor of Safety

Hubbell Power Systems, Inc. recommends a minimum Factor of Safety (FS) for the mechanical strength of the hardware of 2.0.

FS_h	=	2.0 (may be varied based on engineering judgment)	
$R_{W\ ULT}$	=	Minimum ultimate hardware strength requirement based on structural weight	
	=	P_W x FS_h = (39,896 lb) x 2 = 79,792 lb	**Equation 8-3**

Select a pier system with an adequate minimum ultimate strength rating:

$R_{h\ ULT}$	=	86,000 lb - Choose AP-2-UFVL3500.165M[*][14'-0] Modified 2-Piece Pier System	
X_{max}	=	Maximum pier spacing based on hardware capacity	
	=	($R_{h\ ULT}$) / [(FS_h) x (P)]	**Equation 8-4**
	=	(86,000 lb) / [(2) x (4,987)]	
	=	8.6 ft (Use 9.0 ft. Wall and footing are judged able to span this distance)	

Proof Load

Hubbell Power Systems, Inc. recommends a minimum Factor of Safety of 1.5 at installation unless structural lift occurs first.

FS_p	=	Proof Load Factor of Safety[1] = 1.5	**Equation 8-5**
R_p	=	Installation force based on weight of structure to achieve Proof Load verification	
	=	$(FS_p) \times (P_W)$ = (1.5 (8.6 x 4987) = 64,332 lb	
$R_{h\ MAX}$	=	Maximum installation force based on hardware ultimate capacity[2]	
	=	$(Rh_{ULT}/2)\ (1.65) = (86{,}000/2)\ (1.65) = 70{,}950$ lb $R_{W\ MIN} < R_{h\ MAX}$ = OK, where $R_{W\ MIN} = R_p$	

1 Experience has shown that in most cases the footing and stem wall foundation system that will withstand a given long term working load will withstand a pier installation force of up to 1.5 times that long term working load. If footing damage occurs during installation, the free span ($L_{P\ MAX}$) may be excessive.

2 It is recommended that RhMAX not exceed ($R_{h\ ULT}$ / 2) x (1.65) during installation without engineering approval.

Design Recommendations

The result of the analysis provides the following design specifications:

- Underpinning product: ATLAS RESISTANCE® Modified 2-Piece Pier AP-2-UF-3500.165M[*][14'-0]
- Pier spacing: 8.6' on center
- Installation Proof Load: 64,332 lbs ± (unless lift of the structure occurs first)
- Working load is anticipated to be 42,900 lbs ± (4,987 lb/ft x 8.6 ft)
- Anticipated pier depths: 15 ft ±

DESIGN EXAMPLE 2
ATLAS RESISTANCE® PIERS WITH INTEGRATED TIEBACK
SYMBOLS USED IN THIS DESIGN EXAMPLE

kip	Kilopound	8-8
SPT	Standard Penetration Test	8-8
N	SPT Blow Count	8-8
bpf	Blows per Foot	8-8
bgs	Below Ground Surface	8-8
P	Compression Loading	8-8
x	Pier Spacing	8-8
$P_{w\,min}$	Minimum Working Pier Load	8-8
klf	Thousand per Lineal Foot	8-8
DL_h	Horizontal Design Load	8-8
D	Diameter(s)	8-8
c	Cohesion	8-8
φ	Friction Angle	8-8
N_q	Bearing Capacity Factor	8-8
γ	Unit Weight of Soil	8-8
pcf	Pounds per Cubic Foot	8-8
FS	Factor of Safety	8-8
UC_r	Ultimate Tension Capacity	8-8
Q_t	Ultimate Bearing Capacity	8-8
T_u	Ultimate Capacity of Helical Tieback	8-8
A_h	Area of Helix	8-9
K_t	Empirical Torque Factor	8-9
R_p	Proof Load	8-9
FS_p	Proof Load Factor of Safety	8-9
DS	Minimum Installing Force	8-9
$R_{h\,max}$	Maximum Installation Force	8-9
FS_h	Hardware Factor of Safety	8-9

Project Information

An existing three-story commercial building located within a hurricane prone region requires foundation retrofitting for potential scour activity and lateral load forces from hurricane force winds. The structure sits on a shallow foundation system consisting of a 4' high 10" thick stem wall and a 4' wide 12" thick spread footing with three #5 reinforcement bars (Grade 60). The structural Engineer of Record has requested a new foundation system capable of withstanding 2 kips per lineal foot design lateral forces and temporary scour depths to 1' below the existing spread footing. The estimated design compression loading is 5 kips per lineal ft for the existing structure. The structural engineer has determined that the existing foundation system can handle underpinning support spans of 8' or less.

Geotechnical Investigation

A geotechnical investigation was performed to determine the soil types and strengths at the project location. The soil borings advanced near the project location show medium dense silty sand with SPT "N" values ranging from 15 to 25 bpf to a depth of 20 ft bgs. This medium dense silty sand layer is underlain by dense sand and weathered limestone bedrock with SPT "N" values greater than 40 bpf. Groundwater was observed at 18' bgs during the investigation.

DESIGN EXAMPLES

Underpinning System Selection

The availability of a dense stratum with "N" values greater than 40 bpf allows the use of the ATLAS RESISTANCE® Pier. The additional lateral loading can be designed for using a helical tieback anchor and the integrated ATLAS RESISTANCE® Pier bracket. Based on the design compression loading (P) of 5 kips per lineal ft and the allowable pier spacing (x) of 8' the required minimum design capacity of the ATLAS RESISTANCE® Pier ($P_{w\ min}$) is (x) x (P) = 8.0 x 5.0 or 40 kips.

The AP-2-UF-3500.165 system could be used since it has a maximum working (design) capacity of 42.5 kips. However, due to the possibility of scour and subsequent lack of soil support the modified pier with a working capacity of 45.5 kips is recommended (AP-2-UF-3500.165M) with at least three modified pier sections to increase the rotational stiffness of the bracket.

Helical Tieback Design and Installation

With a maximum spacing of 8' and an estimated design lateral line load of 2 klf, the horizontal design load (DL_h) at the tieback anchor location is 16 kips. The tieback anchors are typically installed between 15° to 25° from horizontal. An installation angle of 20o was chosen after determining that there are no underground structures/conduits that may interfere with the tieback installation. The tieback must be designed with a minimum embedment depth of 5D (distance from the last helical plate to the ground surface) where D = diameter of the helical plate. The tieback will be designed to bear in the silty sand with "N" values of 20 bpf observed at 5 to 10 feet bgs. Based on the SPT "N" values and soil descriptions, the following parameters are used in the design:

- Cohesion (c) = 0
- Friction angle (φ) = 34°
- Bearing capacity factor (N_q) = 21
- Unit weight of soil (γ) = 115 pcf

Using a Factor of Safety (FS) = 2 on the design load and an installation angle of 20°, the required ultimate tension capacity of the tieback (UC_r) is (FS x DL_h) / cos 20° = (2 x 16) / cos 20° = 34 kip. The ultimate bearing capacity (Q_t) of a helical tieback can be determined from:

$Q_t = A_n (cN_c + qN_q)$ **Equation 8-6**

Try a Type SS5 series (12”-14” Lead) with a length of 15 ft:

Check depth criteria based on:

- A starting depth of 4 ft below the ground surface
- tieback length of 15 ft
- An installation angle of 20°

The length to the top of the lead helix is 15 ft - 3(12/12) - 4/12 = 11.7 ft. The depth of embedment would be 4 + 11.7sin (20) = 4 ft + 4 ft = 8 ft which is greater than 5D (6 ft), so the depth criteria is met.

Check the ultimate capacity of the helical tieback (T_u) using:

- $N_q = 21$

$$d_{avg} = 4\text{ ft} + \left[15\text{ ft} - \frac{1}{2}\left[\frac{3\,(12\text{in}) + 4\text{ in}}{(12\text{ in/ft})}\right]\right]\sin(20) = 8.6\text{ ft}$$ **Equation 8-7**

- $\gamma' = 115$ pcf
- $\Sigma A_h = A_{12} + A_{14} = 0.77\text{ ft}^2 + 1.05\text{ ft}^2 = 1.82\text{ ft}^2$

$$Q_t = 1.82\text{ ft}^2\,(8.6\text{ ft})(115\text{pcf})(21) = 37.8\text{ kips}$$ **Equation 8-8**

Since the ultimate bearing capacity (37.8 kips) is greater than the required ultimate capacity of 34 kips, the Type SS5 (12”-14”) tieback is acceptable. The average minimum installation torque would be UC_r/K_t or 34,000/10 = 3400 ft-lbs. This minimum installation torque is less than the torque rating of the SS5 and SS125 bar; therefore, either shaft size would be acceptable. K_t = empirical torque factor (default value = 10 for the SS series).

The distance from the assumed “active” failure plane to the 14” helix must be at least 5 times its diameter or 6’-0. Both the minimum length and estimated installation torque must be satisfied prior to the termination of tieback installation.

ATLAS RESISTANCE® Pier Underpinning Installation

Given a design load of 40 kips and the potential for 1 ft of temporary exposed pier section due to scour, use the AP-2-UF-3500.165[M]:

- The AP-2-UF-3500.165M pier has a working (design) load capacity of 45.5 kips. The estimated line load (P) is 5 klf, therefore with a maximum pier c-to-c spacing (x) of 8 ft, the piers will experience a design load (P_w) of 40 kips. The spacing may need to be decreased based upon field conditions.
- Use a minimum 3 modified pier sections (10.5 ft) offset halfway from the inner sleeve sections
- The depth to a suitable stratum for ATLAS RESISTANCE® Pier placement is approximately 20 ft bgs
- Install each pier to a minimum installing force, (Proof Load) $R_p = 1.50 \times P_w$ (estimated Factor of Safety (FS_p) of 1.5 on the design load) which makes the minimum installing force DS=60,000 lbs (based on an 8 ft spacing) or imminent lift, whichever occurs first. The maximum installation force ($R_{h\ max}$) shall not exceed $R_{h\ ULT}/2 \times F_{sh}$ or (91,000/2) x 1.65 = 75,000 lbs (estimated Factor of Safety (FS_h) of 1.65 of the design load for hardware).

DESIGN EXAMPLE 3
HELICAL PILE FOUNDATION FOR NEW CONSTRUCTION

SYMBOLS USED IN THIS DESIGN EXAMPLE

L/W .... Length to Width Ratio 8-10
P .... Total Live Load 8-10
DL .... Dead Load 8-10
LL .... Live Load 8-10
SL .... Snow Load 8-10
FS .... Factor of Safety 8-10
P_w .... Working Pier Load 8-10
x .... Pile Spacing 8-11
Q_t .... Ultimate Pile Capacity 8-11
A .... Area of Helix Plate 8-11
c .... Cohesion of Soil 8-11
N_c .... Bearing Capacity 8-11
T .... Torque 8-11
K_t .... Empirical Torque Factor 8-11

Building Type

- Two story residence
- Slab on grade
- Masonry wall, wood frame
- Width = 30 ft, L/W = 1-1/2

Structural Loads

- Total Live Load on perimeter footing = P **Equation 8-9**
- P = Dead Load (DL) + Live Load (LL) + Snow Load (SL)
- P = 1540 + 346 + 162 = 2,048 lbs/ft (See Tables 4-1 and 4-4 in Section 4 for DL and LL)
- Factor of Safety (FS) = 2.0 (minimum)

Pile Spacing

- Estimated working load (P_w) = (x) x (P) **Equation 8-10**
- Estimated pile spacing (x) = 6.0 ft
- P_w = 6.0 x 2,048 = 12,288 lbs

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CHANCE® Helical Pile Selection

RS2875.203 with 8-10-12 helix configuration

Ultimate Pile Capacity

- $Q_t = (A_8 + A_{10} + A_{12})\ c\ N_c$ **Equation 8-11**

  A_8, A_{10}, A_{12} = Projected area of helical plates

  $A_8 = 0.34\ ft^2$ $A_{10} = 0.53\ ft^2$ $A_{12} = 0.77\ ft^2$

  c = 2,000 psf (based on N=16 – Equation, 5-35)

  N_c = Bearing capacity = 9.0
- Q_t = (1.64) (2,000) (9.0)
- Q_t = 29,520 lb (installation depth is over 20 ft)

Check Q_t

- Conduct Field Load Test (if required per specifications)

Estimate Installation Torque

$T = (P_w \times FS)/K_t$ = (12,288 x 2)/9 = 2,750 ft-lb **Equation 8-12**

K_t = empirical torque factor (default value = 9 for the R2875 series)

The rated installation torque of the RS2875.203 series is 5500 ft-lb, which is greater than the required estimated installation torque of 2,750 ft-lb. (OK)

NOTE: If during installation T = 2,750 ft-lb. is not achieved, then two options are available: (1) reduce pile spacing (x), or (2) change helix configuration to a larger combination, i.e., (10”-12”-14”)

Factor of Safety

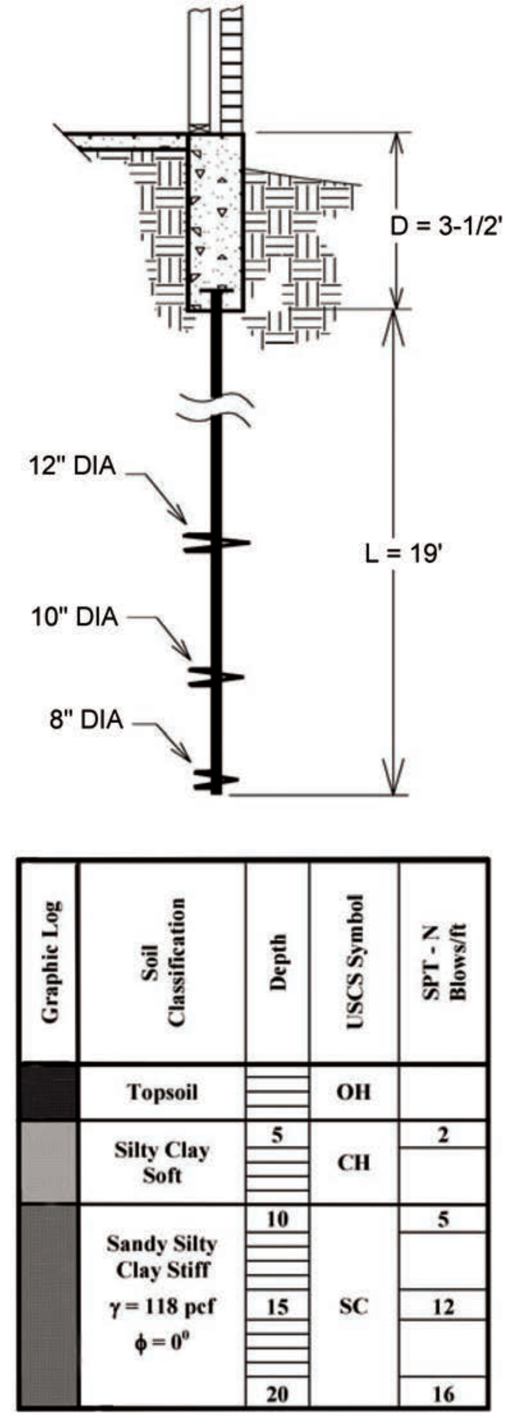
- Theoretical Ultimate Capacity: **Equation 8-13**

  $FS = (Q_t / P_w)$

  FS = 29,520/12,288 = 2.4 (OK)
- Torque Correlation:

  $FS = (T \times Kt)/P_w$

  FS = (2,750 x 9) /12,288 = 2.01 (OK)



Helical Pile Foundation
Figure 8-1

DESIGN EXAMPLES

DESIGN EXAMPLE 4
LIGHT COMMERCIAL STRUCTURE
SYMBOLS USED IN THIS DESIGN EXAMPLE

CH .... Highly Plastic Clay 8-13
PI .... Plasticity Index 8-13
c .... Cohesion of Soil 8-13
γ .... Unit Weight of Soil 8-13
pcf .... Pounds per Cubic Foot 8-13
CL .... Low Plasticity Clay 8-13
SPT .... Standard Penetration Test 8-13
N .... SPT Blow Count 8-13
kip .... Kilopound 8-13
P .... Total Live Load 8-13
P_w .... Working Load 8-13
FS .... Factor of Safety 8-13
U_{Cr} .... Required Ultimate Capacity 8-13
Q_{ult} .... Ultimate Bearing Capacity 8-14
A_h .... Area of Helix 8-14
N_c .... Bearing Capacity 8-14
N_q .... Bearing Capacity Factor 8-14
B .... Footing Width 8-14
φ .... Angle of Internal Friction 8-14
ksf .... Kilo Square Feet 8-14
CMP .... Corrugated Metal Pipe 8-15
DOT .... Department of Transportation 8-15
K_t .... Torque Factor 8-15
T .... Torque 8-16

Problem

Build a new (lightly loaded single story) commercial building on a typical clay soil profile as given on a single boring. The profile consists of the upper 10'-0 of highly plastic clay (CH), Plasticity Index (PI) = 35; cohesion (c) = 2000 psf; unit weight (γ) of 105 pcf. The swell potential of this layer is estimated to be 2". The top 10'-0 layer is underlain by 20' of stiff to very stiff low plasticity clay (CL) that has an Standard Penetration Test (SPT) blow count "N" = 20. The boring was terminated at 30 feet without encountering the water table. No further soil parameters or lab data given.

Possible Solution

Support the structure on a grade beam and structural slab, which is in turn supported by helical piles. Isolate the foundation and slab from the expansive subgrade by forming a 2" void, using a cardboard void form. Assume the water table is at the soil boring termination depth. This is typically a conservative design assumption when the water table is not encountered. The stiff to very stiff clay soil in the 20-foot thick layer is probably at or near 100% saturation (volume of water is the same as the volume of the voids).

Step 1: Feasibility

- Site Access – The site is road accessible, with no overhead or underground obstructions, but the owner is concerned about potential damage to neighboring sites due to vibration and noise.
- Working Loads – The structure is single story, so the working loads are probably considerably less than 100 kip per pile.
- Soils – Boulders, large cobbles, or other major obstructions are not present in the bearing stratum. The clay soil does not appear to be too hard to penetrate with helical piles. See Table 3-1 (Helical Shaft Series Selection) or Figure 3-1 (Product Selection Guide) in Section 3 to determine if helical piles are feasible, and if so, which product series to use.
- Qualified Installers – Local Certified CHANCE® Installers are available and can get competitive bids from a second certified installer 20 miles away.
- Codes – Local building codes allow both shallow and deep foundations.

Cost-bid must be competitive with other systems. Owner may pay a small premium to "protect" the investment in the structure.

Step 2: Soil Mechanics

See Problem section above.

Step 3: Loads

- Exterior Grade Beam – The dead and live loads result in a total live load (P) of 3 kips per lineal foot on the perimeter grade beam (12" wide x 18" deep). The grade beam is designed to span between piles on 8'-0 centers. Therefore, the design or working load per pile (P_W) is 3 kip/ft x 8 ft = 24 kip. A Factor of Safety (FS) of 2.0 is recommended. Therefore, the required ultimate capacity (UC_r) per exterior pile is 24 x 2 = 48 kip compression.
- Interior Columns – The dead load results in 9 kips per column. The live load results in 20 kip per column. The total dead and live load per column is 9 + 20 = 29 kip/column design or working load. A Factor of Safety of 2 is recommended. Therefore, the required ultimate capacity per interior pile is 29 x 2 = 58 kip compression. The required ultimate loads for both the exterior grade beam and interior columns are well within the load ratings of the Hubbell Power Systems, Inc., CHANCE® product series.
- Lateral Loads – The piles are not required to resist any lateral loads.

Step 4: Bearing Capacity

Find the ultimate bearing capacity in the stiff to very stiff clay using hand calculations.

Bearing Capacity: $Q_{ult} = A_h\ (cN_c\ + q'N_q + 0.5\gamma' BN\gamma)$ **Equation 8-14**

For saturated clay soils, the second term of Equation 8-14 becomes zero since the angle of internal friction (φ) is assumed to be zero for saturated clays, thus Nq = 0. The third term (base term) may be dropped because B is relatively small. The simplified equation becomes:

$Q_{ult} = A_h cN_c = A_h c9$ **Equation 8-15**

c (ksf) = N/8 **Equation 8-16**

From Equation 5-35, c (ksf) = 20/8 = 2.5 ksf. At this point, an iterative process is required. Select a helix configuration that is believed can develop the required ultimate capacity. Try a 10"-12" twin helix with a minimum of 5'-0 embedded into the bearing stratum which is the stiff low plasticity clay starting 10 ft below grade. From Table 8-1, the helix area of a 10" helix is 76.4 in^2 or 0.531 ft^2; the helix area of a 12" helix is 111 in^2 or 0.771 ft^2.

Substituting:

$Q_{10} = 0.531\ ft^2 \times 2.5\ ksf \times 9 = 11.95$ kips **Equation 8-17**

$Q_{12} = 0.771\ ft^2 \times 2.5\ ksf \times 9 = 17.35$ kips

$Q_t = \Sigma Q_h = 11.95 + 17.35 = 29.3$ kips

Standard Helix Sizes, Table 8-1

DIAMETER in (cm)	AREA ft^2 (m^2)
6 (15)	0.185 (0.0172)
8 (20)	0.336 (0.0312)
10 (25)	0.531 (0.0493)
12 (30)	0.771 (0.0716)
14 (35)	1.049 (0.0974)

Another trial is required because the total ultimate capacity (Q_t = 29.3 kip) is less than required. Try a three-helix configuration (10"-12"-14") with a minimum of 5'-0 embedded in the bearing stratum. From Table 8-1, the helix area of a 14" helix is 151 in^2 or 1.05 ft^2.

$Q_{14} = 1.05\ ft^2 \times 2.5\ ksf \times 9 = 23.63$ kips **Equation 8-18**

$Q_t = \Sigma Q_h = 11.95 + 17.35 + 23.63 = 52.93$ kips

To achieve the necessary Factor of Safety of 2, two helical piles with a 10"-12" helical configuration can be used under the interior columns (29.3 x 2 = 58.6 ≅ 59 kips ultimate capacity) and a single helical pile with a 10"-12"-14" helical configuration can be used under the perimeter grade beam. The termination of the helical pile in a concrete cap or grade beam should be made with an appropriately designed pile cap or an available "new construction" bracket from Hubbell Power Systems, Inc. This will allow the foundation to rise up, should the swell ever exceed the 2" void allowance, but to shrink back and rest on the pile tops.

Checking Bearing Capacity Using HeliCAP® Engineering Software

A sample tabular data printout is shown in Figure 8-2, where the twin helix (10"-12") Q_{ult} = 29.2 kip ≅ 29.3 kip, OK; and the triple helix (10"-12"-14") Q_{ult} = 52.8 kip ≅ 52.93 kip, OK

Steps 5 and 6: Lateral Capacity and Buckling

- Lateral Capacity – None is required in the statement of the problem. In reality, horizontal loads due to wind will be resisted by net earth pressure (passive-active) on the grade beam and/or caps. See Section 5 for an explanation of earth pressure resistance.
- Buckling Concerns – The soil density and shear strength is sufficient to provide lateral confinement to the central steel shaft. This is supported by the fact that the SPT blow count is greater than four for the top clay layer. Should analysis be required, the Davisson method described in Section 5 may be used to determine the critical load.

Step 7: Corrosion

No electrochemical properties were given for the clay soil. Generally, undisturbed, i.e., non-fill, material tends to be benign as little oxygen is present and the ions that are present in solution are not washed away due to flowing water or fluctuating water level. In the absence of soil data, a useful guide is to observe the use of corrugated metal pipe (CMP) by the local Department of Transportation (DOT). If the DOT uses CMP, the likelihood is that the local soils are not very aggressive.

Step 8: Product Selection

Ultimate capacity for a 10"-12" configuration per Step 4 above was 29 kip, and the ultimate capacity for a 10"-12"-14" configuration was 53 kip. Table 8-2 shows that both CHANCE® Helical Type SS5 and Type RS2875.276 product series can be used, since 53 kip is within their allowable load range. Note that Table 8-2 assumes a Kt of 10 ft-^{1} for the Type SS product series and K_t of 9 ft-^{1} for the Type RS2875 product series. In this case, use the Type SS5 product series because shaft buckling is not a practical concern and the required capacity can be achieved with less installation torque.

Practical Guidelines for Foundation Selection, Table 8-2

INSTALLATION TORQUE	ULTIMATE LOAD[1]		DESIGN LOAD[2]		HELICAL PILE PRODUCT SERIES
	kip	kN	kip	kN	
5,500	55	244	27.5	110	SS5
5,500	49.5	202	24.75	110	RS2875.203
7,000	70	312	35	156	SS150
8,000	72	320	36	160	RS2875.276

[1] Based on a torque factor (K_t) = 10 for SS Series and K_t = 9 for RS2875 Series.
[2] Based on a Factor of Safety of 2.

For the 10"-12" configuration, the minimum depth of 18'-0 can be achieved by using a lead section, which is the first pile segment installed and includes the helix plates, followed by two or three plain extensions. For the 10"-12"-14" configuration, the minimum depth of 21'-0 can be achieved by using a lead section followed by three or four plain extensions. The exact catalog items to use for a specific project are usually the domain of the contractor. Your Certified CHANCE® Installer is familiar with the standard catalog items and is best able to determine which ones to use based on availability and project constraints. For your reference, catalog numbers with product descriptions are provided in Section 7 of this design manual.

The head of the helical pile is to be approximately 1'-0 below grade in the grade beam or cap excavation, which will put the twin-helix pile tip 18'-0 below the original ground level and the three-helix screw foundation tip 21'-0. These are minimum depths, required to locate the helix plates at least 5'-0 into the bearing stratum. On large projects, it is advisable to add 3% to 5% extra extensions in case the soil borings vary considerably or if widely spaced borings fail to indicate differences in bearing depths.

Step 9: Field Production Control

Use K_t = 10 ft^{-1} for CHANCE® Helical Type SS material if verification testing is not done prior to production work. The minimum depth and minimum installing torque must both be achieved. If the minimum torque requirement is not achieved, the contractor should have the right to load test the helical pile to determine if K_t is greater than 10 ft^{-1}. Verification testing is often done in tension since it's simpler and less costly to do than compression testing, and the compressive capacity is generally higher than tension capacity, which results in a conservative site-specific K_t value.

Estimate installing torque for field production control and specifying the minimum allowable without testing.

$Q_{ult} = K_t T$, or $T = Q_{ult}/K_t$ **Equation 8-19**

where: Q_{ult} = UC_r in this example

Interior columns: $T = Q_{ult}/K_t$ = (58,000 lbs/2 piles)/10 ft^{-1} = 2,900 ft-lb ≅ 3,000 ft-lb for the minimum average torque taken over the last three readings.

Perimeter grade beam: $T = Q_{ult}/K_t$ = 48,000 lb/10 ft^{-1} = 4,800 ft-lb for the minimum average torque taken over the last three readings.

Note that the torque rating for the CHANCE® Helical Type SS5 product series is 5,500 ft-lb – OK.

Step 10: Product Specifications

See Section 7, Product Drawings and Ratings and Appendix C for Hubbell Power Systems, Inc. model specifications.

Step 11: Load Test

Since this is a small project with low loads in "normal" soils, it is acceptable to use the torque correlation method as the driving criteria and omit the "optional" load test.

HeliCAP SUMMARY REPORT

Job Name: Design Manual for New Construction

Job Number: Example 2

Boring No: B-1

Anchor Use: Compression

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Water Table Depth: None

Capacity Summary

Anchor Number	Anchor Family	Helix Depth (ft)	Helix Capacity (kips)	Total Anchor Capacity (kips)	Recommended Ultimate Capacity (kips)	Torque (ft-lbs)
Anchor 1	Angle: 90 Datum Depth: 0 Length: 18					
12" helix	SS 5	15	17.3t			
			17.3c	29.2t	29.2t	2925
10" helix	SS 5	17.5	11.9t	29.2c	29.2c	
			11.9c			
Anchor 2	Angle: 90 Datum Depth: 0 Length: 21					
14" helix	SS 5	15	23.6t			
			23.6c			
12" helix	SS 5	18	17.3t			
			17.3c	52.8t	52.8t	5287
10" helix	SS 5	20.5	11.9t	52.8c	52.8c	
			11.9c			

Soil Profile

Top of Layer Depth (ft)	Soil Type	Cohesion (lb/ft2)	N	Angle of Internal Friction (Degrees)	Unit Weight (lb/ft3)
0	Clay	2000	0	0	105
10	Clay	2500	20	0	120

HeliCAP® Summary Report
Figure 8-2

DESIGN EXAMPLES

DESIGN EXAMPLE 5
HELICAL PULLDOWN® MICROPILES for NEW CONSTRUCTION

SYMBOLS USED IN THIS DESIGN EXAMPLE

HPM .... CHANCE HELICAL PULLDOWN® Micropile 8-18
ΣQ_h .... Compression Capacity 8-18
Q_f .... Friction Capacity 8-18
Q_t .... Total Capacity 8-18
D_h .... Diameter of Helix 8-18
PL/AE .... Elastic Compression Line 8-18
N .... Standard Penetration Test Blow Count 8-19
φ .... Angle of Internal Friction 8-19
c .... Cohesion of Soil 8-19

Problem

Determine the capacity of the following CHANCE HELICAL PULLDOWN® Micropile (HPM) installed into the soil described in Figure 8-4.

SS5 1-1/2” x 1-1/2” square shaft

Helix configuration: 8”-10”-12”

Total depth: 40 ft

Grout column: 5” dia x 31 ft

Calculations

End bearing calculations from the HeliCAP® Engineering Software. See Table 8-3 below for the ultimate end bearing capacity of the proposed 8”-10”-12” lead configuration.

Summary: Compression Capacity (ΣQ_h) = 44.7 kip

Summary: Friction Capacity (Q_f) = 22.1 kip (see Table 8-4)

Total Capacity (Q_t) = ΣQ_h + Qf = 44.7 + 22.1 = 66.8 kip

Review of Compression Test

Figure 8-3 is a load deflection plot from the actual compression test on the HPM installed into the soil described in Figure 8-4. From the plotted data, the ultimate capacity (based on 0.08D_h + PL/AE) was 80 kip, compared to the calculated total capacity of 66.8 kip. This calculated value provides a conservative approach to determining the ultimate capacity of an HPM.

HeliCAP® Summary Report, Table 8-3

Job Name: Medina, MN Demonstration

Job Number: Stannard Soil Anchor Systems

Water Table Depth: 15 ft

Boring No: B-1

Anchor Use: Compression

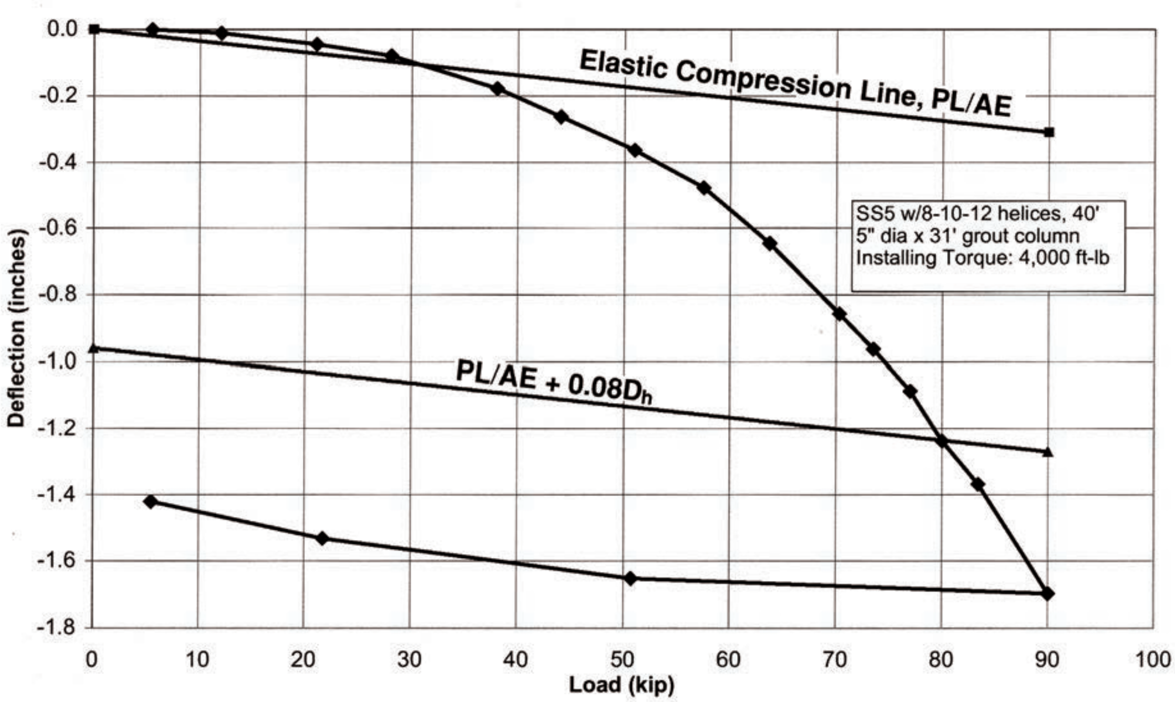
Capacity Summary

Anchor Number	Anchor Family	Helix Depth (ft)	Helix Capacity (kips)	Total Anchor Capacity (kips)	Recommended Ultimate Capacity (kips)	Torque (ft-lbs)
Anchor 1	Angle: 90 Datum Depth: 0 Length: 40					
12" helix	SS 5	35	17.9t 19.9c			
10" helix	SS 5	37.5	14.3t 14.8c	41.9t	41.9t	4263
8" helix	SS 5	39.5	9.6t 9.8c	44.7c	44.7c	

Friction Calculation (See Soil Boring Log in Figure 8-4), Table 8-4

DEPTH (ft)	SOIL	"N"	ESTIMATED COHESION (lb/ft^2)	ESTIMATED φ	EFFECTIVE UNIT WEIGHT (lb/ft^3)	AVERAGE OVERBURDEN (lb/ft^2)	ADHESION/ FRICTION (lb/ft^2)	SIDE FRICTION (lb)
0 -9	CLAY	6	750	-	92	-	682	8040
9 - 15	CLAY	2	250	-	84	-	250	1965
15 - 18	CLAY	1	125	-	20	-	125	491
18 - 22	SAND	5	-	29	23	1438	798	3192
22 - 28	CLAY	7	875	-	32	-	682	5364
28 - 31	SAND	8	-	30	38	1733	1001	3003
							TOTAL	**22055**

Notes: (1) $\varphi = 0.28N + 27.4$ (2) $c = (N \times 1000) / 8$ (3) Area/ft of pile = $\pi \times d = \pi\ (5/12) = 1.31 ft^2/ft$



Helical Pulldown® Micropile Compression Test
Figure 8-3

MINNESOTA DEPARTMENT OF TRANSPORTATION - GEOTECHNICAL SECTION
LABORATORY LOG & TEST RESULTS - SUBSURFACE EXPLORATION

UNIQUE NUMBER

U.S. Customary Units

MINNESOTA DEPARTMENT OF TRANSPORTATION

State Project	Bridge No. or Job Desc.	Trunk Highway/Location	Boring No.	Ground Elevation
		Anchor Demonstration - Medina, MN	1	

Location
Co. Coordinate: X= Y= (ft.)
Latitude (North)= Longitude (West)=

Drill Machine 91
Hammer CME Automatic Calibrated
SHEET 1 of 2
Drilling Completed 7/16/01

DEPTH	Depth / Elev.	Lithology	Classification	Drilling Operation	SPT N60 / REC (%)	MC (%) / RQD (%)	COH (psf) / ACL (ft)	γ (pcf) / Core Breaks	Soil / Rock	Other Tests Or Remarks / Formation or Member
			FILL, mixture of sandy lean clay, clayey sand, a little gravel, brown and gray		8					
5					4					
	9.5		SAPRIC PEAT, black, soft (PT)							
10					2					
	12.0		ORGANIC CLAY, grayish brown, very soft (OH)							
15					WH					
	18.0		SILTY SAND, fine grained, gray, waterbearing, loose, lenses of lean clay (SM)							
20					5					
	22.0		LEAN CLAY, gray, firm (CL)							
25					7					
	28.0		CLAYEY SAND, a little gravel, gray, firm to stiff to very stiff, lenses of sand (SC/CL)							
30					8					
35										

Index Sheet Code 3.0 (Continued Next Page) Soil Class:LR Rock Class: LR Edit: Date 7/19/01
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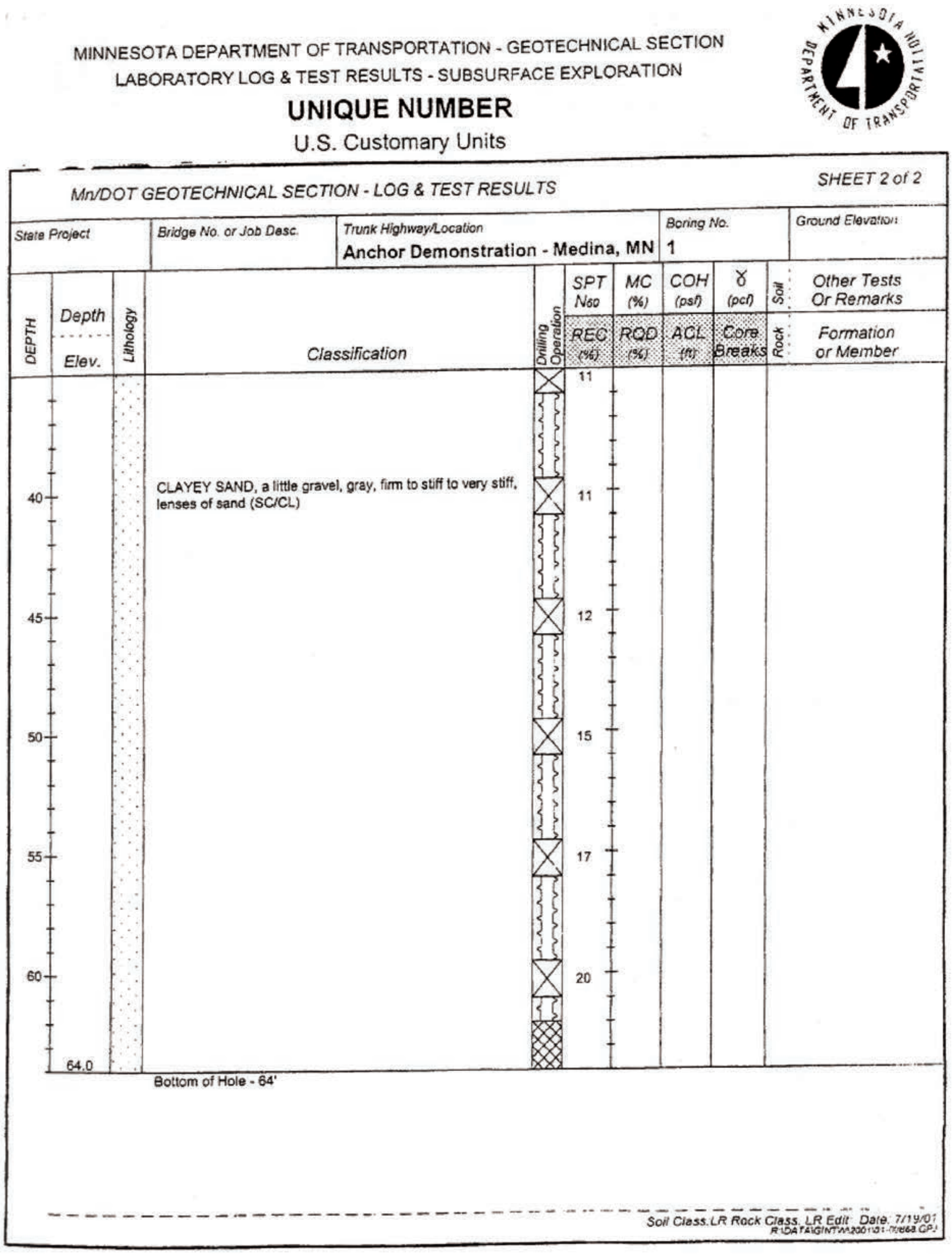
Soil Boring Log
Figure 8-4
(Sheet 1 of 2)



MINNESOTA DEPARTMENT OF TRANSPORTATION - GEOTECHNICAL SECTION
LABORATORY LOG & TEST RESULTS - SUBSURFACE EXPLORATION

UNIQUE NUMBER

U.S. Customary Units

Mn/DOT GEOTECHNICAL SECTION - LOG & TEST RESULTS — SHEET 2 of 2

State Project	Bridge No. or Job Desc.	Trunk Highway/Location	Boring No.	Ground Elevation
		Anchor Demonstration - Medina, MN	1	

DEPTH	Depth / Elev.	Lithology	Classification	Drilling Operation	SPT N60 / REC (%)	MC (%) / RQD (%)	COH (psf) / ACL (ft)	γ (pcf) / Core Breaks	Soil / Rock	Other Tests Or Remarks / Formation or Member
			CLAYEY SAND, a little gravel, gray, firm to stiff to very stiff, lenses of sand (SC/CL)		11					
40					11					
45					12					
50					15					
55					17					
60					20					
	64.0		Bottom of Hole - 64'							

Soil Class.LR Rock Class. LR Edit: Date: 7/19/01

Soil Boring Log
Figure 8-4
(Sheet 2 of 2)

DESIGN EXAMPLE 6
HELICAL PILES for BOARDWALKS

SYMBOLS USED IN THIS DESIGN EXAMPLE

Symbol	Description	Page
SPT	Standard Penetration Test	8-23
N	SPT Blow Count	8-23
WOH	Weight of Hammer	8-23
P_w	Working Pier Load	8-23
FS	Factor of Safety	8-24
UC_r	Required Ultimate Capacity	8-24
Q_h	Ultimate Capacity of Helix Plate	8-24
A	Projected Area of Helix Plate	8-24
D	Vertical Depth to Helix Plate	8-24
γ'	Effective Unit Weight of Soil	8-24
N_q	Bearing Capacity Factor	8-24
K	End Condition Parameter	8-25
P_{crit}	Critical Load	8-25
E	Modulus of Elasticity	8-25
I	Moment of Inertia	8-25
L_u	Unsupported Length	8-25
K_t	Empirical Torque Factor	8-25

Soils

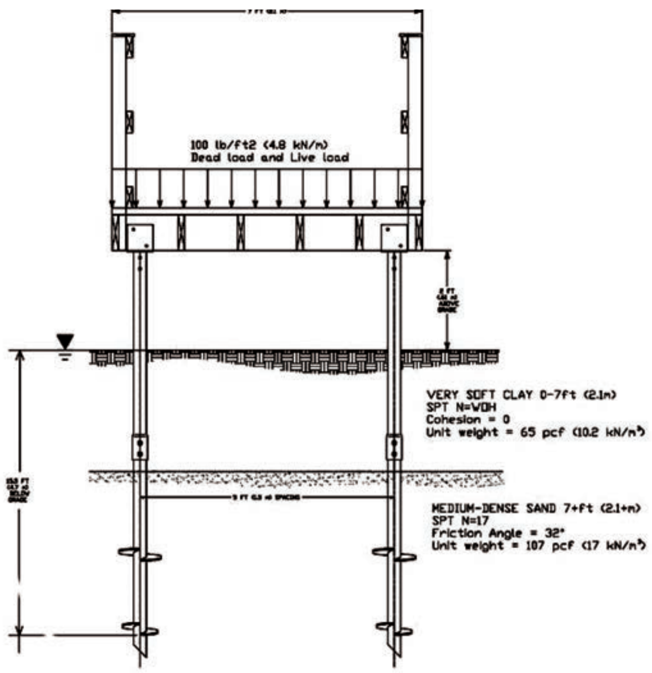
A helical pile foundation is proposed to support a pedestrian walkway. The soil profile consists of 7'-0 (2.1 m) of very soft clay with a reported Standard Penetration Test (SPT) blow count "N" equal to weight of hammer (WOH) and a unit weight of 65 lb/ft^3 (10.2 kN/m^3). Below the very soft clay is a thick layer of medium-dense sand with a SPT blow count value of 17. The correlated friction angle is 32° and the unit weight is 107 lb/ft^3 (16.8 kN/m^3). The water table is located at the surface. The proposed helical pile is connected to the walkway with a CHANCE® Walkway Support Bracket. The helical piles must be checked for lateral stability in the very soft clay.

Walkway

- The helical piles are spaced 5 ft (1.5 m) apart and are exposed 2 ft (0.61 m) above grade as shown in Figure 8-5.
- The walkway is 7 ft (2.1 m) wide; each pile group or "bent" is spaced 10'-0 apart.

Structural Loads

- The dead and live vertical load is 100 lb/ft^2 (4.8 kN/m^2). Lateral loads are negligible.
- The required compression load per helical pile (P_w) is 100 lb/ft^2 x 7'-0 x 10'-0 = 7000 lb/2 helical piles = 3500 lb (15.6 kN) per pile.
- Using a Factor of Safety (FS) of 2, the required ultimate capacity (UC_r) per helical pile is 3500 lb x 2 = 7000 lb (31.1 kN).



Helical Piles for Boardwalks
Figure 8-5

CHANCE® Helical Pile Selection

- Try a twin-helix configuration with 10” (254 mm) and 12” (305 mm) diameters.
- Try either Type SS5 1-1/2” (38 mm) Square Shaft or Type RS2875.203 2-7/8” (73 mm) Round Shaft material.

Ultimate Pile Capacity

The top-most helix should be at least three diameters into a suitable bearing soil; which in this example is the medium-dense sand starting 7 ft (2.1 m) below grade. The spacing between helix plates is also three diameters; which is 3 x 10” = 2.5 ft (0.8 m) for a 10”-12” (254 mm – 305 mm) configuration. Finally, the distance from the bottom-most helix to the pile tip is 0.5 ft (0.15 m). Therefore, the minimum overall length for a 10”-12” helix configuration in this soil profile is 7 ft + (3 x 12 inch) + 2.5 ft + 0.5 ft = 13 ft (4 m). The effective unit weight is the submerged unit weight in this case, because the water table is at the ground surface. The general bearing capacity equation (simplified for cohesionless soils) is:

$$Q_h = AD\gamma' N_q \qquad \textbf{Equation 8-20}$$

where:

Q_h	=	Ultimate capacity of helix plate
A	=	Projected area of helix plate
D	=	Vertical depth to helix plate
γ'	=	Effective unit weight of soil = 2.6 lb/ft³ (0.4 kN/m³) for the very soft clay and 44.6 lb/ft³ (7.1 kN/m³) for the medium-dense sand
N_q	=	Bearing capacity factor for cohesionless soils = 17 for 32° sand

For a 10”-12” configuration, the bearing capacity equation is:

$$\Sigma Qh = A_{10}D_{10}\gamma' N_q + A_{12}D_{12}\gamma' N_q \qquad \textbf{Equation 8-21}$$

where:

$$\Sigma Qh = 0.531\ ft^2[(7\ ft \times 2.6\ lb/ft^3) + (5.5\ ft \times 44.6\ lb/ft^3)]17 + 0.771\ ft^2[(7\ ft \times 2.6\ lb/ft^3) + (3\ ft \times 44.6\ lb/ft^3)]17$$

$$\Sigma Qh = 4371\ lb\ (19.4\ kN)$$

4371 lb is less than the required ultimate capacity (7000 lb) needed for the vertical piles. Greater capacity can be obtained by extending the helix plates deeper into the medium-dense sand. Try extending the pile length 3 ft (0.9 m) deeper so that the tip is 16 ft (4.9 m).

ΣQh	=	$0.531\ ft^2[(7\ ft \times 2.6\ lb/ft^3) + (8.5\ ft \times 44.6\ lb/ft^3)]17$ $+ 0.771\ ft^2[(7\ ft \times 2.6\ lb/ft^3) + (6\ ft \times 44.6\ lb/ft^3)]17$	**Equation 8-22**
ΣQh	=	7332 lb (32.6 kN)	

7332 lb is greater than the required ultimate capacity needed for the vertical piles, so 16 ft (4.9 m) pile length will work.

Buckling

Check for buckling on Type SS5 1-1/2" (38 mm) square shaft and Type RS2875.203 2-7/8" (73 mm) OD pipe shaft material with 2 ft (0.61 m) of exposed shaft above grade. Assume a free-fixed (K = 2) end-condition. Assume the very soft clay provides no lateral support, i.e., the pile shaft is unsupported above the sand, so the unsupported (effective) length (L_u) of the "column" is 2 ft + 7 ft = 9 ft (2.7 m).

Euler's Equation: $P_{crit} = \pi^2 EI/[KL_u]^2$

For Type SS5 square shaft material:

P_{crit}	=	$\pi^2\ [30\times10^6\ lb/in^2\ ][.396\ in^4]/[2 \times 108\ in]^2$	**Equation 8-23**
P_{crit}	=	2513 lb (11.2 kN)	

The critical load for the Type SS5 series is less than the required 7000 lb (31.1 kN) ultimate capacity, so a shaft with greater stiffness is required.

For Type RS2875.203 pipe shaft material:

P_{crit}	=	$\pi^2[30\times10^6\ lb/in^2\ ][1.53\ in^4]/[2 \times 108\ in]^2$	**Equation 8-24**
P_{crit}	=	9710 lb (42.2 kN)	

The critical load for Type RS2875.203 pipe shaft is greater than the required 7000 lb (31.1 kN) ultimate capacity. Use the RS2875.203 series (2-7/8 inch (73 mm) OD pipe shaft material).

Torque

Torque required	=	Required ultimate capacity/K_t	**Equation 8-25**
where:	=	K_t = 9 (26) for RS2875 round shaft	
Torque required	=	7000 lb / 9	
Torque required	=	778 ft-lb (1186 N-m)	

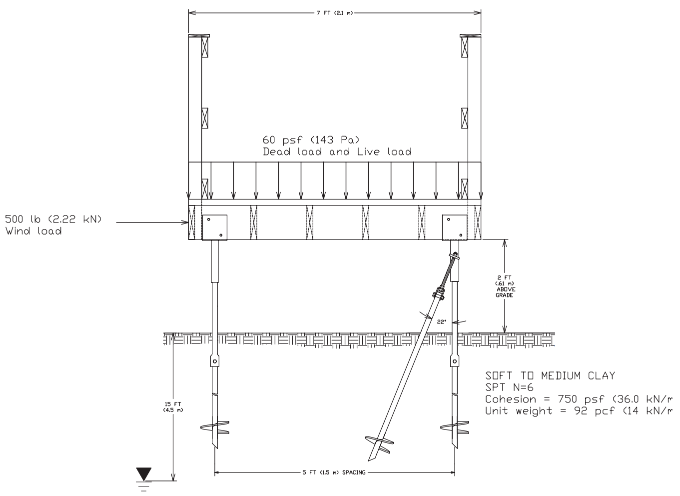
The torque strength rating for RS2875.203 material is 5,500 ft-lb (7,500 N-m) - OK.

DESIGN EXAMPLES

DESIGN EXAMPLE 7
HELICAL PILES for BOARDWALKS with LATERAL SUPPORT
SYMBOLS USED IN THIS DESIGN EXAMPLE

SPT .... Standard Penetration Test 8-26
N .... SPT Blow Count 8-26
psf .... Pounds per Square Foot 8-26
GWT .... Ground Water Table 8-26
FS .... Factor of Safety 8-26
UC_r .... Required Ultimate Capacity 8-26
Q_t .... Total Capacity 8-27
A .... Area of Helix 8-27
c .... Cohesion of Soil 8-27
N_c .... Bearing Capacity 8-27
P_{crit} .... Critical Load 8-27
K_t .... Empirical Torque Factor 8-27

A CHANCE® Helical Type SS5 square shaft is proposed as the foundation for a pedestrian walkway. The pier is connected to the walkway with a CHANCE® Helical Walkway Support Bracket with lateral support. The soil is a soft to medium clay with a Standard Penetration Test (SPT) "N" value of 6, cohesion of 750 psf (36.0 kN/m^2) and unit weight of 92 lb/ft^3 (14 kN/m^3). The ground water table (GWT) is 15 ft (4.5 m) below grade.



Helical Piles for Boardwalks
with Lateral Support
Figure 8-6

Walkway:

- The piles are spaced 5 ft (1.5 m) apart and are exposed 2 ft (0.61 m) above grade.
- The walkway is 7 ft (2.1 m) wide and pier sets are 5 ft (1.5 m) apart.
- The battered pile is at an angle of 22°.

Structural Loads:

- Using a Factor of Safety (FS) of 2, the required ultimate capacity (UC_r) per vertical pile is 4550 lb (20 kN).
- Using a Factor of Safety of 2, the required ultimate capacity (UC_r) per battered pile is 2646 lb (12 kN).

CHANCE® Helical Pile Selection:

- Try a Type SS5 square shaft with a 12" (305 mm) diameter helix.

CHANCE® Helical Pile Selection

- Try a Type SS5 square shaft with a 12" (305 mm) diameter helix.

Ultimate Pile Capacity:

The pile depth needs to be at least 5 diameters into the soft to medium clay layer. Therefore the vertical pile length should be at least 5 ft (1.5 m) below grade.

$$Q_t = AcN_c \qquad \textbf{Equation 8-26}$$

$$Q_t = [.771\ ft^2][750\ psf][9]$$

$$= 5{,}204\ lb\ (23\ kN)$$

where: A = Projected area of helical plates

c = Cohesion of soil

N_c = Bearing capacity

5,204 lb is greater than UC_r for the vertical pile. The battered pile depth needs to be at least 5 diameters below grade. Therefore the battered pile length should be 6 ft (1.8 m) below grade.

Buckling:

Check for buckling on the SS5 square shaft with 2 ft (0.61 m) of exposed shaft above grade. Assume a pin-pin (K = 1) connection.

Euler's Equation:

$$P_{crit} = \pi^2 EI/[KL_u]^2 \qquad \textbf{Equation 8-27}$$

$$P_{crit} = \pi^2[30x10^6][.396]/[1 \times 24]^2$$

$$P_{crit} = 203{,}354\ lb\ (904\ kN)$$

The critical load is greater than the ultimate vertical load so buckling is not a concern.

Torque:

Torque required = Required load/K_t **Equation 8-28**

where: = K_t = 10 (33) for square shaft

Torque required = 5,204 lb / 10

Torque required = 520 ft-lb (705 N-m)

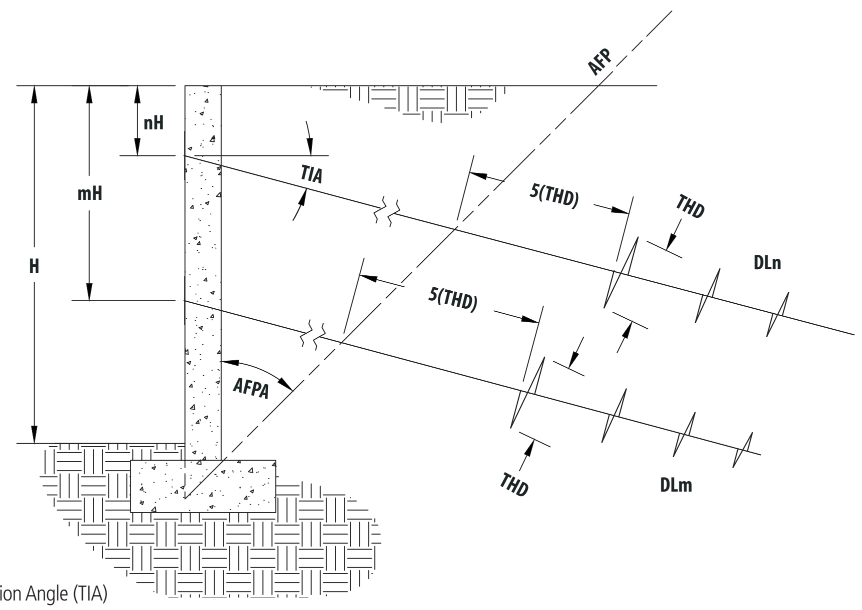
This does not exceed the SS5 torque rating of 5,500 ft-lb (7,500 N-m).

DESIGN EXAMPLES

DESIGN EXAMPLE 8
HELICAL TIEBACK ANCHORS IN CLAY

SYMBOLS USED IN THIS DESIGN EXAMPLE

H	Height of Wall	8-29
nH	Height of Upper Anchor	8-29
mH	Height of Lower Anchor	8-29
GWT	Ground Water Table	8-29
DL_N	Design Load for Upper Anchor	8-29
DL_M	Design Load for Lower Anchor	8-29
Q_{tn}	Ultimate Tension Capacity for Upper Anchor	8-30
Q_{tm}	Ultimate Tension Capacity for Lower Anchor	8-30
A	Area of Helix Plate	8-30
N_c	Bearing Capacity Factor	8-30
c	Cohesion of Soil	8-30
T_u	Ultimate Capacity of Anchors	8-30
FS	Factor of Safety	8-30
T_N	Installation Torque for Upper Anchor	8-30
T_M	Installation Torque for Lower Anchor	8-30
K_t	Empirical Torque Factor	8-30



Tieback Installation Angle (TIA)
Top Helix Diameter (THD)
Assumed Failure Plane (AFP)
Assumed Failure Plane Angle (AFPA)

Helical Tieback Anchor
Figure 8-7

Structure Type

- Cast concrete retaining wall
- Height (H) = 18 ft, thickness = 2'-0
- nH = 0.25H = 4.5 ft, mH = 0.63H = 11.3 ft
- Residual soils: stiff clay with N = 28. No ground water table (GWT) present.
- Tieback installation angle = 15°

Structural Design Loads (See Figure 4-6 in Section 4)

- $DL_N/ft = (12 \times H^2) / \cos 15°$
- $DL_N/ft = (12 \times 18^2)/ \cos 15°$
- $DL_N/ft = 4{,}025$ lb/lin ft
- $DL_M/ft = (18 \times H^2) / \cos 15°$
- $DL_M/ft = (18 \times 18^2)/ \cos 15°$
- $DL_M/ft = 6{,}040$ lb/lin ft

CHANCE® Helical Product Selection

- Wall height ≥ 15 ft; use two rows of tiebacks
- Try Type SS150 series, C150-0169 (8"-10"-12" Lead) for DL_N.
- Try Type SS175 series, C110-0247 (8"-10"-12"-14" Lead) for DL_M.

Ultimate Tension Capacity (Using Bearing Capacity Approach)

Q_{tn}	=	$(A_8 + A_{10} + A_{12}) \times (c\, N_c)$	**Equation 8-29**
A_8, A_{10}, A_{12}	=	Projected area of helical plates (8",10", and 12")	
N_c	=	Bearing capacity factor related to the residual soil, clay	
A_8	=	$0.336\ ft^2$	
A_{10}	=	$0.531\ ft^2$	
A_{12}	=	$0.771\ ft^2$	
N_c	=	9	
c	=	N / 8 = 28 / 8 = 3.5 ksf or 3,500 psf *(see Equation 5-35)*	
Q_{tn}	=	(0.336 + 0.531 + 0.771) x 3,500 x 9	
Q_{tn}	=	51,600 lbs	

Q_{tm}	=	$(A_8 + A_{10} + A_{12} + A_{14}) \times (cN_c)$	**Equation 8-30**
$A_8, A_{10}, A_{12}, A_{14}$	=	Projected area of helical plates (8",10",12", and 14")	
A_{14}	=	$1.049\ ft^2$	
Q_{tm}	=	(0.336 + 0.531 + 0.771+ 1.049) x 3,500 x 9	
Q_{tm}	=	84,640 lbs	

Check Ultimate Anchor Capacity (T_u)

Compare Q_{tN} and Q_{tM} to field load tension tests if required by specifications.

Tieback Spacing

$Spacing_N$ = $(Q_{tN} / FS) / DL_N$ = (51,600 / 2) / (4,025) = 6.4 ft

$Spacing_M$ = $(Q_{tM} / FS) / DL_M$ = (84,640 / 2) / (6,040) = 7.0 ft
(use 6'-6" center to center spacing for both rows of tiebacks)

where: FS = 2.0

Estimate Installation Torque

$T = (DL \times Spacing \times FS) / K_t$ **Equation 8-31**

$T_N = (DL_N \times Spacing_N \times FS) / K_t$ = (4,025 x 6.5 x 2) / 10 = 5,300 ft-lb

$T_M = (DL_M \times Spacing_M \times FS) / K_t$ = (6,040 x 6.5 x 2) / 10 = 7,850 ft-lb

where: K_t = Empirical torque factor (default value = 10 for Type SS series)

Check Installation Torque Ratings

The rated installation torque of the Type SS150 series is 7,000 ft-lbs, which is greater than the required installation torque (T_N) of 5,300 ft-lbs.

The rated installation torque of the Type SS175 series is 10,500 ft-lbs, which is greater than the required installation torque (T_M) of 7,850 ft-lbs.

Minimum Tieback Length

The distance from the assumed "active" failure plane to the 12" helix must be at least 5 x its diameter or 5'-0. The distance from the assumed "active" failure plane to the 14" helix must be at least 5 x its diameter or 6'-0. Both the minimum length and estimated installation torque must be satisfied prior to the termination of tieback installation.

DESIGN EXAMPLE 9
HELICAL TIEBACK ANCHORS IN SAND

SYMBOLS USED IN THIS DESIGN EXAMPLE

φ	Angle of Internal Friction	8-31
γ	Unit Weight of Soil	8-31
pcf	Pounds per Cubic Foot	8-31
K_a	Active Earth Pressure Coefficient	8-31
DL	Design Load	8-31
DL_t	Tieback Design Load	8-31
Q_t	Ultimate Tension Capacity	8-32
A	Area of Helix Plate	8-32
N_q	Bearing Capacity Factor	8-32
Q_t	Total Capacity	8-32
T_u	Ultimate Anchor Capacity	8-32
FS	Factor of Safety	8-32
T	Installation Torque	8-32
K_t	Empirical Torque Factor	8-32

DESIGN EXAMPLES

Structure Type

- Cast concrete retaining wall
- Granular backfill for wall $\varphi = 35°$ $\gamma = 120$ pcf
- Height = 15 ft, thickness =1-1/2 ft
- Anchor Height = 1/3H = 5 ft
- Residual soils: silty coarse sand; medium to dense $\varphi = 31°$ $\gamma = 118$ pcf. No ground water table present.
- Tieback installation angle = 25°

Structural Design Loads

- Use backfill $\varphi = 35°$
- $K_a = (1 - \sin\varphi) / (1 + \sin\varphi) = 0.27$
- DL/ft = $(1/2\ \gamma\ H^2\ K_a) / \cos 25°$
  $= [1/2\ (120)\ (15)^2\ (0.27)] / \cos 25°$
  = 4,000 lb/lin ft
- Assume tieback carries 80%; therefore, $DLt\ /_{ft} = 0.80 \times 4{,}000. = 3{,}200$ lb/lin ft

CHANCE® Helical Product Selection

- Wall height ≤ 15 ft; use single row of tiebacks
- Try Type SS5 series, C1500007 (8”-10”-12” Lead)

Ultimate Tension Capacity (Using Bearing Capacity Approach)

Q_t	=	$(A_8 + A_{10} + A_{12}) \times (q_h N_q)$	**Equation 8-32**
A_8, A_{10}, A_{12}	=	Projected area of helical plates (8", 10" and 12")	
N_q	=	Bearing capacity factor related to φ of residual soil (31°)	
A_8	=	$0.336\ ft^2$	
A_{10}	=	$0.531\ ft^2$	
A_{12}	=	$0.771\ ft^2$	
N_q	=	15 (from Equation 5-19)	
q_h	=	$\gamma \times D_h$ (depth of helix below ground line, ft)	
q_8	=	118 pcf (5’ + 25’ sin 25°) = 1836 psf	
q_{10}	=	118 pcf (5' + 23' sin 25°) = 1736 psf	
q_{12}	=	118 pcf (5’ + 20.5’ sin 25° = 1612 psf	
Q_t	=	[(0.336 x 1836) + (0.531 x 1736) + (0.771 x 1612)] x 15	
Q_t	=	41,725 lbs	

Check Ultimate Anchor Capacity (T_u)

Compare Q_t to field load tension tests if required by specifications.

Tieback Spacing

where:	$Spacing_N$	=	$(Q_t / FS) / DL_t$ = (41,725 / 2) / (3,200) = 6.5 ft (use 6’-6 center to center spacing)	**Equation 8-33**
	FS	=	2.0	

Estimate Installation Torque

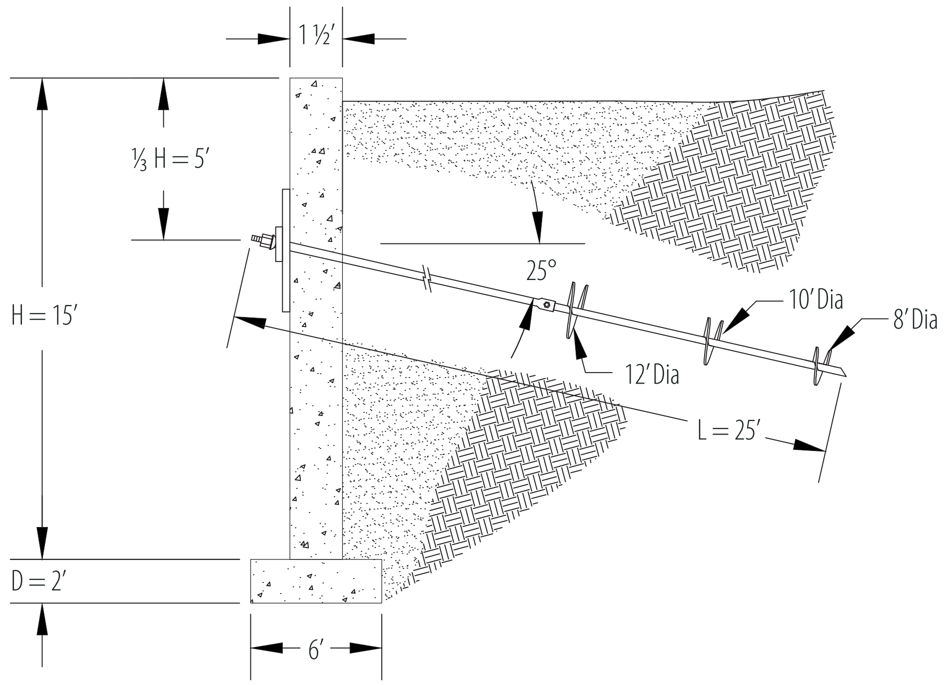
where:	T	=	$(DL_t \times \text{spacing} \times FS) / K_t$ = (3,200 x 6.5 x 2.0) / 10 = 4,200 ft-lb	**Equation 8-34**
	K_t	=	Empirical torque factor (default value = 10 for Type SS series)	

Check Installation Torque Ratings

The rated installation torque of the Type SS5 series is 5,500 ft-lbs, which is greater than the required installation torque (T) of 4,200 ft-lbs.

Minimum Tieback Length

The distance from the assumed “active” failure plane to the 12” helix must be at least 5 times its diameter or 5’-0. Both the minimum length and estimated installation torque must be satisfied prior to the termination of tieback installation.



Helical Tieback Anchor
Figure 8-8

SOIL BORING LOG				
Graphic Log	**Soil Classification**	**Depth**	**USCS Symbol**	**SPT - N Blows/ft**
	Topsoil		OH	
	Silty Sand	5	SM	17
	Silty Coarse Sand γ = 118 pcf ϕ = 31°	10	SM	30
		15		32
		20		34

Soil Boring Log
Figure 8-9

DESIGN EXAMPLE 10
SOIL SCREW® RETENTION WALL SYSTEM
SYMBOLS USED IN THIS DESIGN EXAMPLE

S_V	Vertical SOIL SCREW® Spacing	8-35
S_H	Horizontal SOIL SCREW® Spacing	8-35
L	Length of SOIL SCREW® Anchor	8-35
FS	Factor of Safety	8-35
γ	Unit Weight of Soil	8-35
φ	Internal Angle of Friction	8-35
pcf	Pounds per Cubic Foot	8-35
psf	Pounds per Square Foot	8-35
Ω	Ohms	8-35
ppm	Parts per Million	8-35
GWT	Ground Water Table	8-36
H	Height of Wall	8-36
K_a	Active Earth Pressure Coefficient	8-36
F_1	Horizontal Force from Retained Soil	8-36
F_2	Horizontal Force from Surcharge Load	8-36
L_x	Horizontal Length of SOIL SCREW® Anchor	8-37
e	Eccentricity of Vertical Force	8-37
σv	Vertical Stress	8-37
Q_{allow}	Allowable Bearing Capacity	8-37
kip	Kilopound	8-38
N_q	Bearing Capacity Factor	8-39
P	Ultimate Tension Capacity	8-39
A	Area of Helix	8-39
y	Difference in Depth of SOIL SCREW® Anchor from End to End	8-39
θ	Angle of SOIL SCREW® Anchor (from horizontal)	8-39
psi	Pounds per Square Inch	8-40
ksi	Kilopounds per Square Inch	8-40
d	Diameter of Welded Fabric Wire	8-40
D	Diameter of Rebar	8-40
A_s	Area of Steel	8-40
m_v	Vertical Moment Resistance	8-41

T_{FN} .... Maximum Helical Anchor Head Load 8-41
C_F .... Facing Pressure Factor 8-41
V_N .... Punching Shear Strength of Facing 8-41
f'_c .... Compressive Strength of Concrete 8-41
h_c .... Thickness of Facing 8-41
D'_c .... Effective Cone Diameter at Center of Facing 8-41
$FS_{internal}$ .... Internal Factor of Safety 8-42
FS_{global} .... Global Factor of Safety 8-43
M_c .... Cantilever Moment 8-43
FS_{MC} .... Factor of Safety for Cantilever Moment 8-44
S_c .... Shear Force 8-44
FS_{shear} .... Factor of Safety for Shear Force 8-44

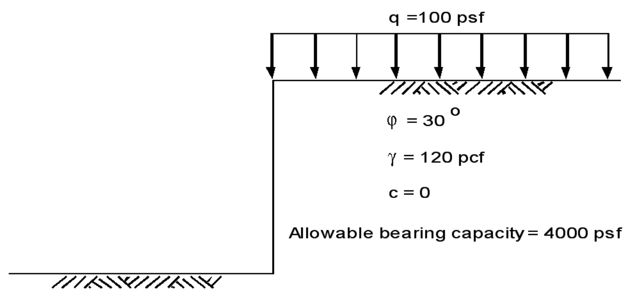
Problem

Determine the SOIL SCREW® Anchor spacing (S_V, S_H), SOIL SCREW® Anchor length (L) and facing requirements for an excavation support system for a 23 foot deep excavation in a silty sand. The required design Factor of Safety (FS) for internal stability is 1.5, and for global stability is 1.3.

Step 1 - Define Design Parameters

Given: The unit weight (γ) and friction angle (φ) of the silty sand is 120 pcf and 30° respectively. The allowable bearing capacity of the silty sand at the bottom of the excavation is 4000 psf. The electrochemical properties of the silty sand are listed below:

Resistivity	4000 Ω/cm
pH	7
Chlorides	50 ppm
Sulfates	100 ppm



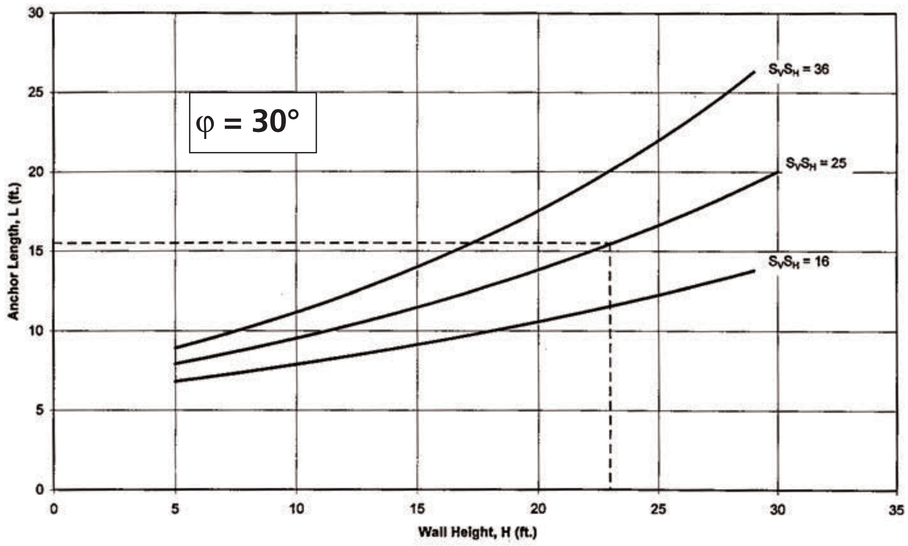
Excavation Profile
Figure 8-10

A design live surcharge load of 100 psf is considered to be applied uniformly across the ground surface at the top of the wall. The wall face is vertical. Groundwater is located 60 feet below the ground surface.

CHANCE® Type SS5 Helical SOIL SCREW® Anchors, for which lead sections and extensions are available in 5' and 7' lengths, are to be used for the SOIL SCREW® Anchors. The design life of the structure is one year. Design SOIL SCREW® Anchor lengths will be governed by the lead and extension pieces and thus will be 10', 12', 14', 15', 17', 19', etc.

Step 2 - Check the Preliminary Feasibility of the SOIL SCREW® Retention Wall System

The medium dense, silty sands at this site are well suited for the SOIL SCREW® Retention Wall System (i.e., good stand up time). The ground water table (GWT) is well below the bottom of the excavation. The conditions at the site are therefore favorable for the SOIL SCREW® Retention Wall System.



Preliminary Design Chart
Figure 8-11

Design charts are used to determine preliminary SOIL SCREW® Anchor spacing and lengths for the given wall geometry, loading and soil conditions. For the soil conditions, φ = 30°, enter the Preliminary Design Chart (Figure 8-11) along the x-axis at a wall height (H) = 23 ft. A typical SOIL SCREW® Anchor spacing for soils with "good" stand up time is 5 ft. x 5 ft. Therefore, use the S_VS_H = 25 curve to determine the preliminary SOIL SCREW® Anchor length (L) = 16 ft.

Step 3 - Determine External Earth Pressures

Use Equation 8-35 to determine the active earth pressure (K_a) at the back of the reinforced soil mass.

$K_a = \tan^2 [\,45 - (\varphi/2)]$ **Equation 8-35**

$K_a = \tan^2 [\,45 - (30/2)] = 0.33$

Step 4 - Check Preliminary SOIL SCREW® Anchor Length with Respect to Sliding

Available SOIL SCREW® Anchor lengths for CHANCE® Helical Type SS5 anchors are 10', 12', 14', 15', 17', 19', etc. The 16 foot preliminary length determined in Step 2 does not account for surcharge loading, which tends to increase SOIL SCREW® Anchor lengths. Try 19' SOIL SCREW® Anchors (length to height ratio of 0.83). For preliminary designs for walls with the given soil and loading conditions, a length to height ratio of 0.8 to 1.0 is a starting point for the analysis and appears to be conservative.

The horizontal force from the retained soil (F1) is determined using Equation 8-36.

$F_1 = 1/2\ K_a\ \gamma\ H^2$ **Equation 8-36**

$F_1 = 1/2\ (0.33)\ (120)\ 23^2 = 10474$ lb/lf of wall

The horizontal force from the surcharge load (F2) is determined using Equation 8-37.

$F_2 = K_a\ qH = 0.33\ (100)\ 23 = 759$ lb/lf of wall **Equation 8-37**

Using 19' SOIL SCREW® Anchors installed at a 15° angle, the horizontal length (L_X) of the SOIL SCREW® Anchor is determined using Equation 8-38.

$$L_x = L \cos 15° \qquad \textbf{Equation 8-38}$$

$$L_x = 19 \cos 15° = 18.4 \text{ ft}$$

The Factor of Safety against sliding is determined using Equation 8-39.

$$FS = \frac{\gamma H L_x \tan \varphi}{F_1 + F_2} = \frac{120\ (23)\ 18.4 \tan 30}{10474 + 759} \qquad \textbf{Equation 8-39}$$

$$FS = 2.61$$

Step 5 - Check Required Bearing Capacity at the Base of the Wall

Determine the eccentricity (e) of the resultant vertical force using Equation 8-40.

$$e = \frac{[F1\ (H/3)] + [F2\ (H/2)]}{\gamma H L_x} \qquad \textbf{Equation 8-40}$$

$$= \frac{[10474\ (23/3)] + [759\ (23/2)]}{120\ (23)\ 18.4}$$

$$= 1.75 < (L_x/6) = (18.4/6) = 3.06$$

The vertical stress (σ_v) of the bottom of the wall is determined using Equation 8-41.

$$\sigma_v = \frac{\gamma H L_x + q L_x}{L_x - 2e} = \frac{120\ (23)\ 18.4 + 100\ (18.4)}{18.4 - 2\ (1/75)} = 3532 \text{ psf} \qquad \textbf{Equation 8-41}$$

Given the allowable bearing capacity (Q_{allow}) is 4000 psf:

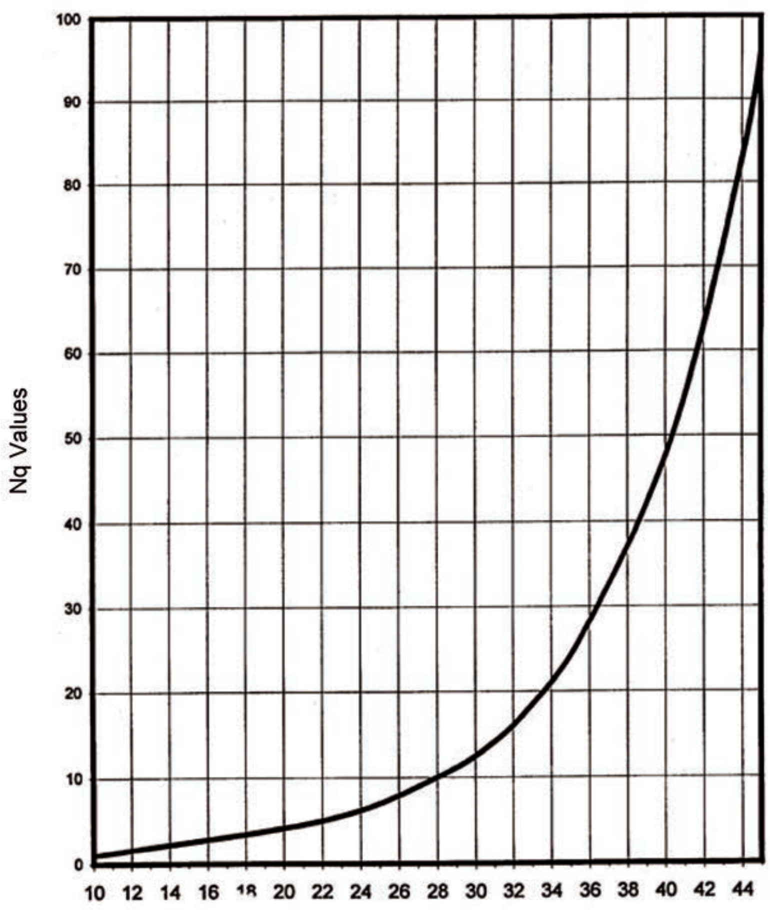
$$Q_{allow} = 4000 \text{ psf} > \sigma_v = 3532 \text{ psf} \qquad \textbf{Equation 8-42}$$

DESIGN EXAMPLES

Step 6 - Determine the Allowable Helical Anchor Strength

Allowable Design Strength of Type SS5 Helical Anchor (Service Life = 75 Years), Table 8-5

Ta 75 yrs (kips)	V 75 yrs (kips)	ALLOWABLE DESIGN STRENGTH (TEMPORARY STRUCTURES) (kips)	ALLOWABLE DESIGN STRENGTH 75 yrs (kips)
50	37	45	37

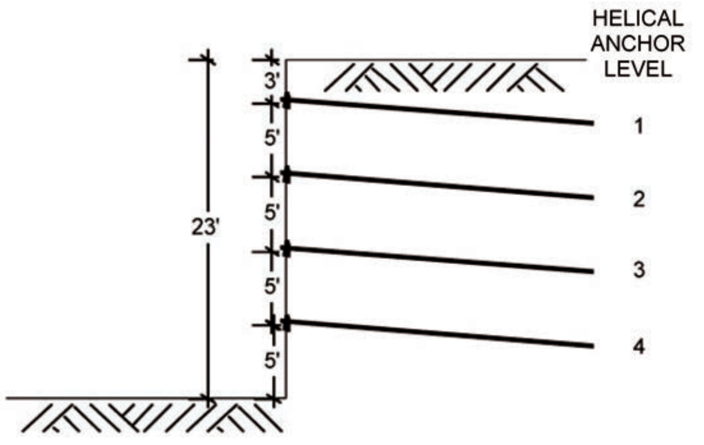


Angle of Internal Friction φ

Bearing Capacity Factor Nq vs Soil Friction Angle φ
Figure 8-12

The SOIL SCREW® Anchor wall is a temporary structure with a design life of one year. From Table 8-5, the allowable design strength of the CHANCE® Helical SS5 Anchor is 45 kips. This table is based on the following electrochemical properties of soil:

Resistivity:	>3000 Ω/cm
pH:	>5<10
Chlorides:	100 ppm
Sulfates:	200 ppm
Organic content:	1% max



Helical Anchor Levels
Figure 8-13

Step 7 - Estimate the Tension Capacity of the SOIL SCREW® Anchors

Determine the bearing capacity factor (N_q) for helical anchors for a sand with an effective friction angle, $\varphi = 30°$. From Figure 8-12, $N_q = 14$. Assumed vertical spacing is 5 feet (see Figure 8-13). Nail pattern is as shown in Figure 8-13. There are eight helices per anchor, as shown in Figure 8-14.

The ultimate tension capacity (P) of the Helical SOIL SCREW® Anchor at Level 1 is determined using Equation 8-43.

$$P = \sum_{i=1}^{8} A_i q_i N_q \qquad \textbf{Equation 8-43}$$

Helical anchors have 8" diameter helixes. The helix area (A) can be calculated using Equation 8-44.

$$A = \pi (0.33)^2 \qquad \textbf{Equation 8-44}$$
$$= 0.336 \text{ ft}^2 \text{ (use } 0.34 \text{ ft}^2\text{)}$$

The ultimate tension capacities for the helical anchors at the various levels are determined using Equation 8-45.

$$y = L (\sin \theta) \qquad \textbf{Equation 8-45}$$
$$= 19 (\sin 15^{o})$$
$$= 4.9 \text{ ft}$$

where:

L = Length of SOIL SCREW® Anchor

θ = Installation angle (from horizontal)

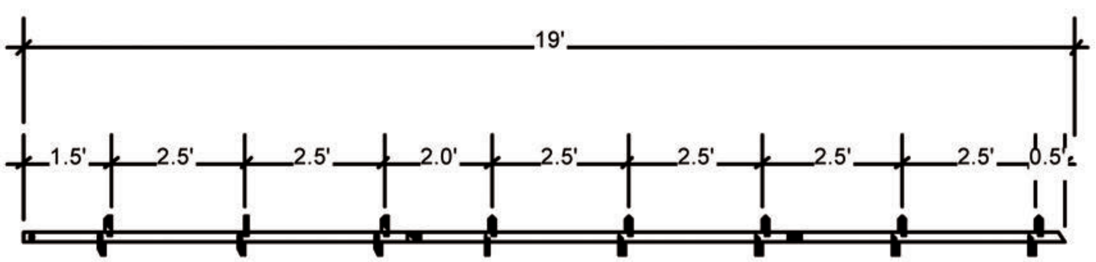
Average Overburden Depth = 3 + (y/2) = 5.5 ft at Level 1

P_{LEVEL1} = 8 (0.34) 5.5 (120) 14 = 25 kips

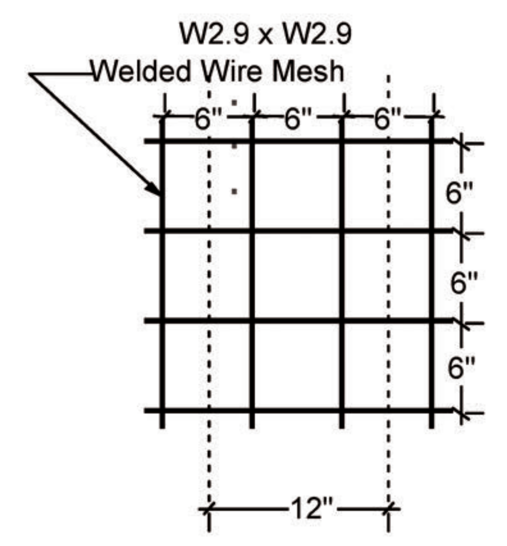
P_{LEVEL2} = 8 (0.34) 10.5 (120) 14 = 48 kips

P_{LEVEL3} = 8 (0.34) 15.5 (120) 14 = 71 kips

P_{LEVEL4} = 8 (0.34) 20.5 (120) 14 = 94 kips



Helical Anchor Helix Spacing
Figure 8-14



Welded Wire Mesh
Figure 8-15

Step 8 - Define a Trial Facing System

Try a 4” thick, 4000 psi shotcrete face with 6 x 6, W2.9 x W2.9 welded wire mesh reinforcing and two #4 vertical rebars at the helical anchor locations. Try a helical anchor spacing of 5 feet vertically and horizontally and an 8” square by 3/4” thick bearing plate with a steel yield stress of 36 ksi.

Step 9 - Determine the Allowable Flexural Strength of the Facing

For typical helical anchor wall construction practice, the facing is analyzed using vertical strips of width equal to the horizontal anchor spacing. For facing systems involving horizontal nail spacings that are larger than the vertical spacing or unit horizontal moment capacities that are less than the vertical unit moment capacities, horizontal strips of width equal to the vertical anchor spacing should be used.

The area of steel (A_S) for a vertical beam of width 5 feet (S_H = 5 feet) with the anchor on the beam’s centerline is determined using Equation 8-46. Diameter (d) of the welded fabric wire is 0.192”. Diameter (D) of the rebar is 0.500”. For a 5 foot wide vertical beam centered between the anchors, the rebars are located at the beam edges and should be ignored. A_S is calculated using Equation 8-47. The corresponding average nominal unit moment resistances are determined using Equation 8-48.

Equation 8-46

$$A_{s,neg} = \left(\frac{pd^2}{4}\right)\left(\frac{in^2}{wire}\right) \times \left(\frac{2\ wires}{ft}\right)(5\ ft)\ \left(\frac{pD^2}{4}\right)\left(\frac{in^2}{rebar}\right) \times \left(\frac{2\ rebars}{5\ ft}\right)(5\ ft)$$

$$= \left(\frac{p\,(0.192^2)}{4}\right)(2)\,(5)\ \left(\frac{p\,(0.500^2)}{4}\right)(2)$$

$$= 0.682\ in^2$$

Equation 8-47

$$A_{s,pos} = \left(\frac{\pi D^2}{4}\right)\left(\frac{in^2}{wire}\right) \times \left(\frac{2\ wires}{ft}\right)(5\ ft)$$

$$= \frac{\pi\,(0.192)^2}{4}\ (2)\,(5)$$

$$= 0.289\ in^2$$

Equation 8-48

$$m_v = \frac{A_s F_y \left(d - \frac{A_s F_y}{1.7 f'_c b}\right)}{b}$$

$$m_{v,neg} = \frac{0.682\,(60)\left(2 - \frac{0.682\,(60)}{1.7\,(4)\,(5 \times 12)}\right)}{5\,(12)}$$

$$= 1.30k - in/in$$

$$= 1.30k - ft/ft$$

$$m_{v,pos} = \frac{0.289\,(60)\left(2 - \frac{0.289\,(60)}{1.7\,(4)\,(5 \times 12)}\right)}{5\,(12)}$$

$$= 0.566\ k\text{-}ft/ft$$

Step 10 - Determine the Maximum Helical Anchor Head Load

Determine the maximum helical anchor head load that will produce the allowable moments determined in Step 9 using Equation 8-49. Using Table 8-6, determine the facing pressure factor (C_F) for temporary shotcrete facing 4" thick.

$$T_{FN,\ flexure} = C_F\ (m_{v,neg} + m_{v,pos})\ 8\ (SH/SV) \qquad \textbf{Equation 8-49}$$

$$T_{FN,\ flexure} = 2.0\ (1.30 + 0.57)\ 8\ (5\ ft/5\ ft) = 29.8\ kips$$

Facing Pressure Factor, Table 8-6

NOMINAL FACING THICKNESS (in)	TEMPORARY FACING C_F	PERMANENT FACING C_F
4	2.0	1.0
6	1.5	1.0
8	1.0	1.0

Step 11 - Determine the Allowable Punching Shear Strength of the Facing

The punching shear strength (V_N) is determined using Equation 8-50.

$$V_N = 0.125\ \sqrt{f'_c}\ \ pD'_c\ \ h_c \qquad \textbf{Equation 8-50}$$

$$V_N = 0.125\ \sqrt{4}\ \pi\ (12)\ (4) = 38\ kips$$

where:

f'_c = 4,000 psi = 4 ksi

h_c = 4 in

D'_c = 8 + 4 = 12 in

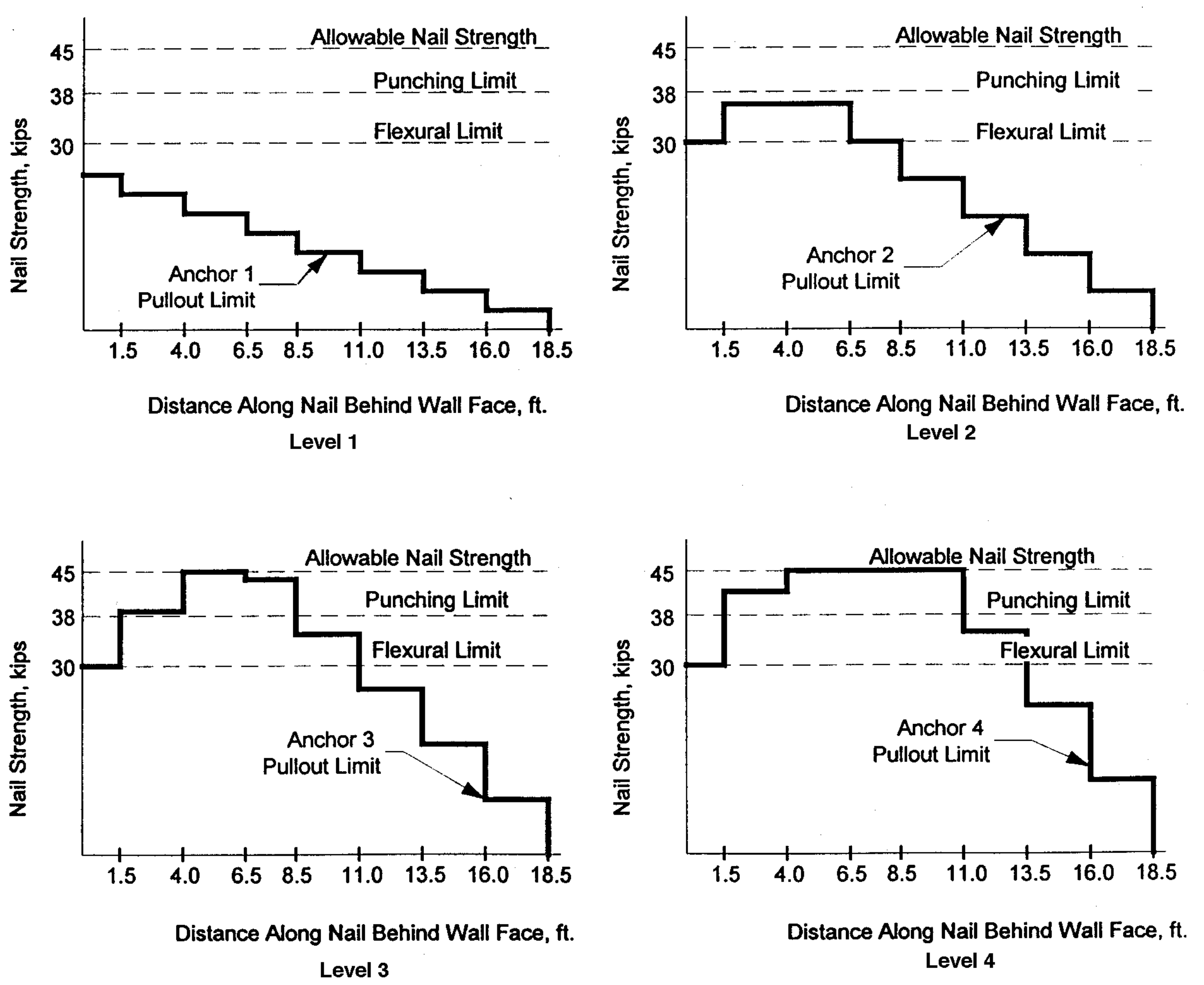
Step 12 - Determine Critical Helical Anchor Head Load for Punching

Determine the critical helical anchor head load (T_{FN}) for punching using Equation 8-51.

$$T_{FN,\ punching} = V_N = 38\ kips \qquad \textbf{Equation 8-51}$$

Step 13 - Construct SOIL SCREW® Anchor Strength Envelope

Construct the strength envelope at each anchor level as shown in Figure 8-16. At the wall face, the anchor head flexural strength is less than the anchor head punching strength and therefore controls. There are eight helices per anchor. Each step in strength equals the single-helix bearing capacity for the anchor layer (Step 7). From the last helix (working from right to left) increase the pullout capacity in a stepwise fashion. If the pullout envelope working from the back of the nail does not intersect the flexural limit line, the strength envelope will look like that shown for Anchor 1. If the pullout envelope working from the back of the nail exceeds the flexural limit, then construct a pullout envelope working from the flexural limit at the head of the nail.



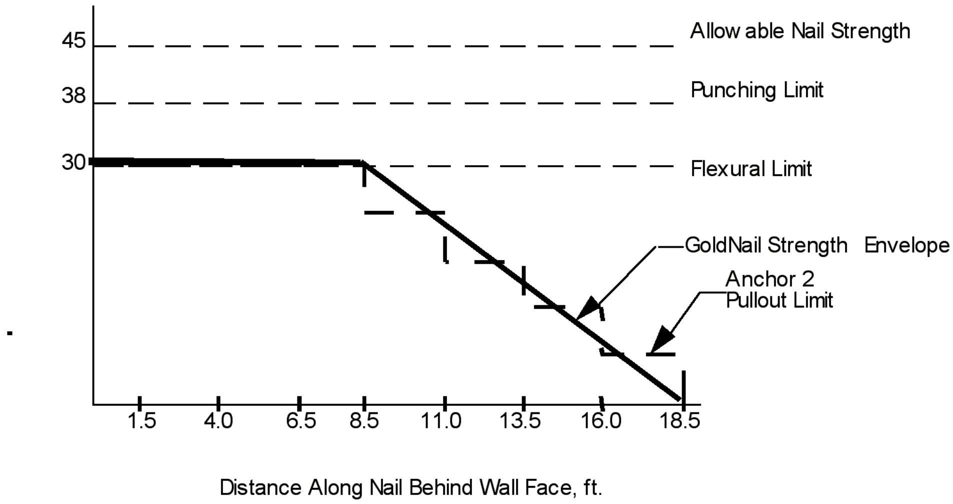
Anchor Pullout Limits
Figure 8-16

Step 14 - Evaluate Internal and Compound Stability

GoldNail 3.11, "A Stability Analysis Computer Program for Soil Nail Wall Design," developed by Golder and Associates, was used to perform the internal and compound stability analysis. Refer to Attachment EX1 in the CHANCE® SOIL SCREW® Retention Wall System Design Manual for printout result of this stability analysis. The following discussion is based on these results.

The anchor strength envelope developed in Step 13 needs to be modified for GoldNail. The increase in pullout capacity along the length of the nail is estimated for GoldNail as straight lines, not step functions. An example of this modification for Anchor Level 2 is shown in Figure 8-17.

Within GoldNail there are several analysis options. The option used for this example is "Factor of Safety." Using this option, the Internal Factor of Safety ($FS_{internal}$) = 2.11 for the anchor pattern defined in Step 7. The GoldNail output printout lists "Global Stability" not "Internal Stability." However, the location of the critical failure surface (Circle #13) indicates an internal mode of failure, as shown on the GoldNail geometry printout.



GoldNail Strength Envelope
Figure 8-17

Step 15 - Check Global Stability

Analysis was performed for the given slope geometry by the computer program PCSTABL6H, developed by Purdue University and modified by Harald Van Aller, and the pre-processor STED, developed by Harald Van Aller. The resulting Global Factor of Safety (FSglobal) = 1.93. Refer to Attachment EX2 in the CHANCE® SOIL SCREW® Retention Wall System Design Manual for printout results of this global stability analysis.

Step 16 - Check Cantilever at Top of Wall

In Step 7 the layout of anchors was assumed. The cantilever at the top of the wall from Step 7 is 3 feet. Check cantilever moment (Mc) using Equation 8-52.

Equation 8-52

$$M_c = K_a\gamma\left[\left(\frac{H_1^2}{2}\right)\left(\frac{H_1}{3}\right) + q\left(\frac{H_1^2}{2}\right)\right]$$

$$= 0.33\,(120)\left[\left(\frac{3^2}{2}\right)\left(\frac{3}{3}\right) + 100\left(\frac{3^2}{2}\right)\right]$$

$$= 326.7\text{lb - ft/ft}$$

Maximum allowable moment at midspan (Step 9) is 566 lb-ft/ft., therefore:

FS_{MC} = (566 / 327) = 1.73 OK **Equation 8-53**

Check shear force at cantilever (Sc) using Equation 8-54.

$$= K_a [\, \gamma (H_1^2 / 2) + qH_1 \,]$$ **Equation 8-54**

$$S_c = 0.33 [\, 120 (3^2 / 2) + 100 (3) \,]$$

$$= 277 \text{ lb/ft}$$

Determine allowable shear using Equation 8-55

$$V_N = 0.125 \sqrt{f'_c}\ h_c$$ **Equation 8-55**

$$= 0.125 \sqrt{4}\ (4) = 1000 \text{ lb/lf}$$

FS_{shear} = (1000 / 277) = 3.6 OK **Equation 8-56**

DESIGN EXAMPLES

DESIGN EXAMPLE 11
HELICAL PILES/ANCHORS for TELECOMMUNICATION TOWERS

SYMBOLS USED IN THIS DESIGN EXAMPLE

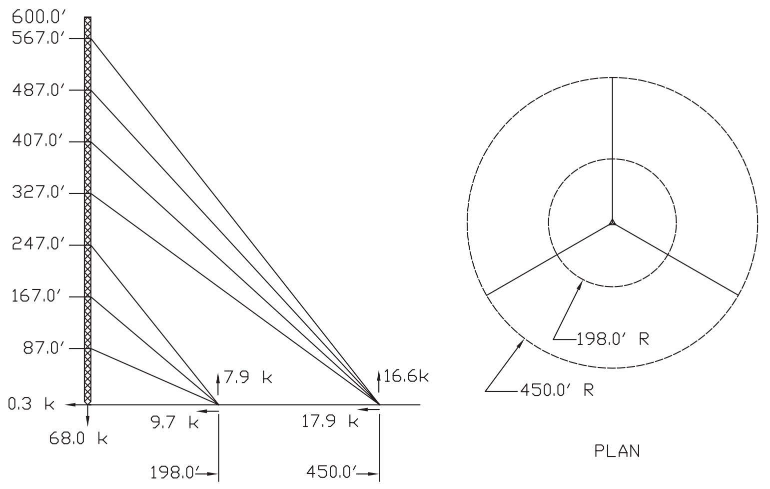
SST	Self-Supporting Tower	8-45
T_{ug}	Upper Guywire Anchor Tension	8-46
IA_{ug}	Upper Guywire Installation Angle	8-46
T_{lg}	Lower Guywire Anchor Tension	8-46
IA_{lg}	Lower Guywire Installation Angle	8-46
C	Compression	8-46
V	Horizontal Shear	8-46
FS	Factor of Safety	8-46
kip	Kilopound	8-46
R_{uc}	Recommended Ultimate Capacity	8-46
K_t	Empirical Torque Factor	8-46
T	Minimum Installation Torque	8-46
DL	Resultant Axial Load	8-47

Purpose

This Design Example provides an aid in the selection of appropriate helical guywire anchors and center mast helical piles for telecommunication towers.

The guywire loads are to be resisted by a helical tension anchor. When the vertical and horizontal components are provided the resultant must be determined as well as the angle between the resultant load and the horizontal, (this is the angle the helical anchor should be installed at to properly resist the guywire load(s)). There may be one or more guywires that come to the ground to be restrained by one or more helical anchors depending on the magnitude of the load and/or the soil strength. Helical piles can be used to resist the loads from the structure mast. These loads will generally be composed of a vertical load and a lateral load at the base of the mast or pole.

If the structure is a self supporting tower (SST), the loads from each leg of the tower must be resisted. These generally consist of vertical uplift and compression loads and a horizontal shear load at the ground line. These three loads can be dealt with in a number of ways. Typically one or more helical piles are used for each leg of the tower and may be installed at a batter to better resist the horizontal shear loads. Steel grillages and reinforced concrete caps have been used to facilitate load transfer from the structure to the helical piles. This type design will not be covered in this design example since the intent is to focus on the guyed mast tower structure.



Tower Guy Anchor and Foundation
Figure 8-18

Figure 8-18 shows the tower that will be used for these sample calculations. It will be noted that the four upper guywires come to the ground at a single guywire point and that the three lower guywires come to ground at a different guywire point. There must be at least a single helical anchor installed at each of these points to provide restraint for the guywires which in turn stabilize the tower by resisting lateral loads on the structure.

For this tower, the vertical and horizontal components of the guywire loads are given and must be resolved into the tension load the helical guywire anchor is to resist.

Upper Guywire Loads

- Vertical load component = 16.6 k
- Horizontal load component = 17.9 k
- Tension in the upper guywire anchor = T_{ug} = $(16.6^2 + 17.9^2)^{0.5}$ = 24.4 k
- Helical guywire anchor installation angle = IA_{ug} = $\tan^{-1}$ (16.6/17.9) = 43°

Lower Guywire Loads

- Vertical load component: 7.9 k
- Horizontal load component: 9.7 k
- Tension in the lower guywire anchor = T_{lg} = $(7.9^2 + 9.7^2)^{0.5}$ = 12.5 k
- Helical guywire anchor installation angle = IA_{lg} = $\tan^{-1}$ (7.9/9.7) = 39°

Mast Foundation Loads

- Compression (C) = 68.0 k
- Horizontal shear (V) = 0.3 k

Selecting Helical Guywire Anchors

Hubbell Power Systems, Inc. HeliCAP® Engineering Software will be utilized to determine the appropriate helical anchor/pile sizes for this tower. Soil conditions are shown in the Sample Boring Log in Figure 8-19. The soil data and guywire anchor data was input into the HeliCAP® Engineering Software to get an appropriate output. The minimum acceptable Factor of Safety (FS) = 2.

Upper Guywire Helical Anchor

The HeliCAP® Summary Report for the upper guywire helical anchor is shown in Figure 8-20. This report provides the following information:

- Helical Anchor: SS5 (1.5" square shaft, 5500 ft-lbs torque rating, 70 kips ultimate tension rating)
- Lead Section: 4 helix (8"-10"-12"-14")
- Installation Angle: 43°
- Datum Depth (depth below grade where installation starts): 0 ft
- Length: 45 (ft along the shaft at the 43° installation angle)
- Recommended Ultimate Capacity (R_{uc}): 50.2t (kips tension)

The Factor of Safety for this tension anchor is R_{uc}/T_{lg} = 50.2 / 24.4 = 2.05 > 2 (OK). Use this helical anchor at each of three upper guywire anchor locations per tower.

The required average minimum installation torque (T) is:

$$T = (T_{ug} \times FS) / K_t \qquad \textbf{Equation 8-57}$$

$$= (24{,}400 \times 2.0) / 10$$

$$= 4{,}900 \text{ ft-lbs}$$

where: K_t = Empirical torque factor = 10 (default value for Type SS5 series)

T = 4,900 ft-lbs is less than the rated torque (5,500 ft-lbs) of the Type SS5 series. (OK).

Lower Guywire Helical Anchor

The HeliCAP® Summary Report for the lower guywire helical anchor is shown in Figure 8-21. This report provides the following information:

- Helical Anchor: SS5 (1.5" square shaft, 5500 ft-lbs torque rating, 70 kips ultimate tension rating)
- Lead Section: 4 helix (8"-10"-12"-14")
- Installation Angle: 39°
- Datum Depth (depth below grade where installation starts): 0 ft
- Length: 25 ft (along the shaft at the 39° installation angle)
- Recommended Ultimate Capacity (R_{uc}): 26.6t (kips tension)

The Factor of Safety for this tension anchor is $R_{uc} / T_{ug} = 26.6 / 12.5 = 2.12 > 2$ (OK) Use this helical anchor at each of three lower guywire anchor locations per tower.

	T	=	$(T_{lg} \times FS) / K_t$	**Equation 8-58**
		=	$(12{,}500 \times 2.0) / 10$	
		=	2,500 ft-lbs	
where:	K_t	=	Empirical torque factor = 10 (default value for Type SS5 series)	

T = 2,500 ft-lbs is less than the rated torque (5,500 ft-lbs) of the Type SS5 series. (OK).

Helical Pile

Given:

- Compression Load = 68.0 k
- Shear Load = 0.3 k

Assume three helical piles installed at 120° intervals in plan view with each pile battered away from vertical at a 10° angle:

68/3 piles = 22.67k ultimate/pile element.

Assume entire shear (0.3 k) is taken by one battered pile. Therefore, the resultant axial load (DL) to a battered pile is:

$DL = (22.67^2 + 0.3^2)^{0.5} = 22.7k$

The HeliCAP® Summary Report for the helical piles is shown in Figure 8-22. This report provides the following information:

- Helical Pile: SS175 (1.75" square shaft, 10,500 ft-lbs torque rating, 100 kips ultimate tension rating)
- Lead Section: 4 helix (8"-10"-12"-14")
- Installation Angle: 80° below horizontal (10° away from vertical)
- Datum Depth: (depth below grade where installation starts): 0 ft
- Length: 34 ft (along the shaft at the 80° installation angle)
- Recommended Ultimate Capacity (R_{uc}): 50.7c (kips compression)

The Factor of Safety for this compression pile is R_{uc} / DL = 50.7 / 22.7 = 2.23 > 2 (OK) Use three SS175 helical piles per tower base. The three helical piles must be captured in a "pile cap." This may be a reinforced concrete cap, the design of which is beyond the scope of this design example. The design of this concrete pile cap is left to the structural engineer.

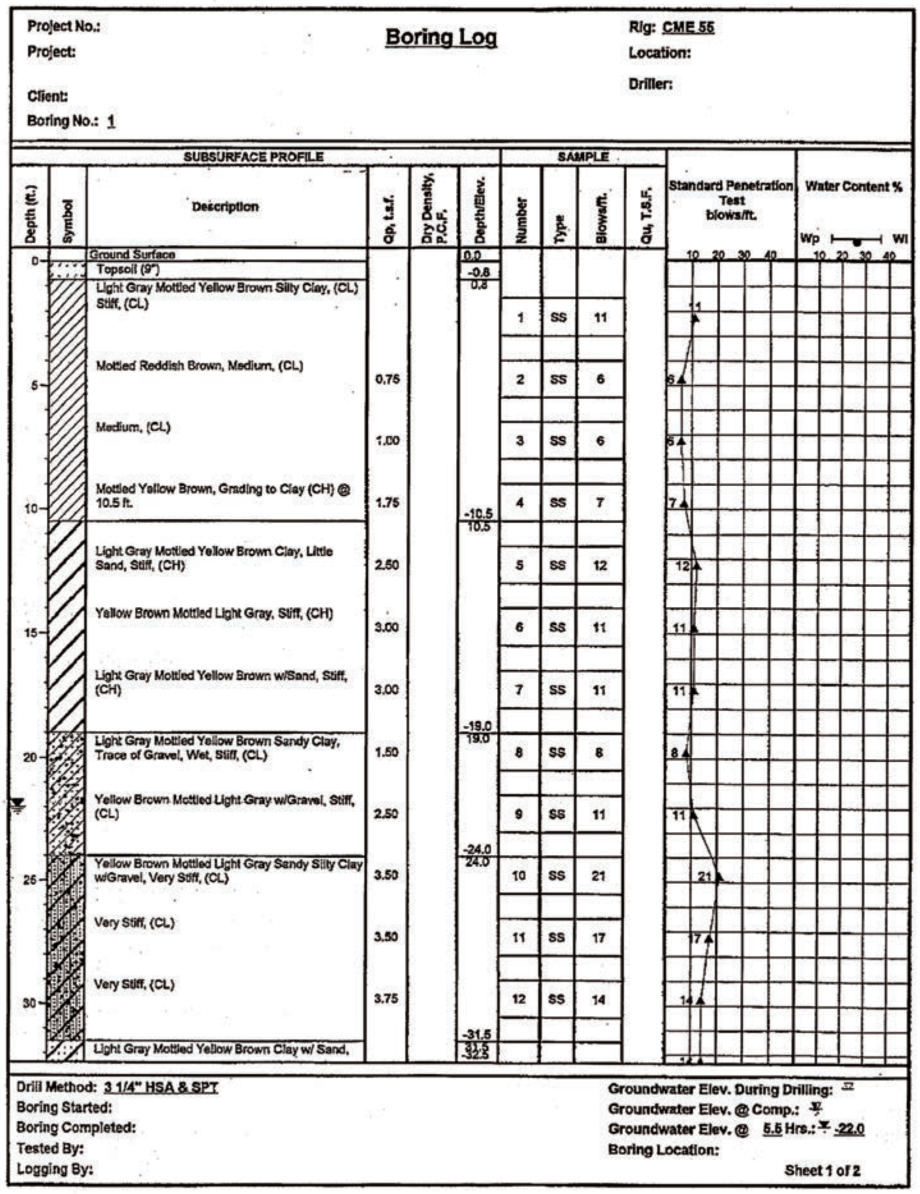
$$T = (DL \times FS) / K_t \qquad \textbf{Equation 8-59}$$

$$= (22{,}700 \times 2.0) / 10$$

$$= 4{,}500 \text{ ft-lbs}$$

where: K_t = Empirical torque factor = 10 (default value for Type SS175 series)

T = 4,500 ft-lbs is less than the rated torque (10,500 ft-lbs) of the Type SS175 series. (OK).



Boring Log

Project No.:
Project:
Client:
Boring No.: 1

Rig: CME 55
Location:
Driller:

Depth (ft.)	Symbol	Description (SUBSURFACE PROFILE)	Qp, t.s.f.	Dry Density, P.C.F.	Depth/Elev.	Number (SAMPLE)	Type	Blows/ft.	Qu, T.S.F.	Standard Penetration Test blows/ft. (10 20 30 40)	Water Content % (Wp — Wl; 10 20 30 40)
0		Ground Surface			0.0						
		Topsoil (9")			-0.8 / 0.8						
		Light Gray Mottled Yellow Brown Silty Clay, (CL) Stiff, (CL)				1	SS	11		11	
5		Mottled Reddish Brown, Medium, (CL)	0.75			2	SS	6		6	
		Medium, (CL)	1.00			3	SS	6		6	
10		Mottled Yellow Brown, Grading to Clay (CH) @ 10.5 ft.	1.75		-10.5 / 10.5	4	SS	7		7	
		Light Gray Mottled Yellow Brown Clay, Little Sand, Stiff, (CH)	2.50			5	SS	12		12	
15		Yellow Brown Mottled Light Gray, Stiff, (CH)	3.00			6	SS	11		11	
		Light Gray Mottled Yellow Brown w/Sand, Stiff, (CH)	3.00		-19.0 / 19.0	7	SS	11		11	
20		Light Gray Mottled Yellow Brown Sandy Clay, Trace of Gravel, Wet, Stiff, (CL)	1.50			8	SS	8		8	
		Yellow Brown Mottled Light Gray w/Gravel, Stiff, (CL)	2.50		-24.0 / 24.0	9	SS	11		11	
25		Yellow Brown Mottled Light Gray Sandy Silty Clay w/Gravel, Very Stiff, (CL)	3.50			10	SS	21		21	
		Very Stiff, (CL)	3.50			11	SS	17		17	
30		Very Stiff, (CL)	3.75			12	SS	14		14	
		Light Gray Mottled Yellow Brown Clay w/ Sand,			-31.5 / 31.5 / 32.5						

Drill Method: 3 1/4" HSA & SPT
Boring Started:
Boring Completed:
Tested By:
Logging By:

Groundwater Elev. During Drilling:
Groundwater Elev. @ Comp.:
Groundwater Elev. @ 5.5 Hrs.: -22.0
Boring Location:

Sheet 1 of 2

Sample Boring Log
Figure 8-19

HeliCAP SUMMARY REPORT

b Name: Tower Guy Calculations
b Number: Upper Guy
ıring No: 1
ıchor Use: Tension

C:\Documents and Settings\jlgoen\Desktop\Tow
6/1/2006 8:43:36 AM
Water Table Depth: 22 ft

Capacity Summary

Anchor Number	Anchor Family	Helix Depth (ft)	Helix Capacity (kips)	Total Anchor Capacity (kips)	Recommended Ultimate Capacity (kips)	Torque (ft-lbs)
Anchor 1	Angle: 43 Datum Depth: 0 Length: 45					
14" helix	SS 5	25.2	16.9t 24.8c			
12" helix	SS 5	27.2	17t 14.7c			
10" helix	SS 5	28.9	10.1t 9.5c	50.2t	50.2t	5502
8" helix	SS 5	30.3	6.1t 5.3c	54.4c	54.4c	

Soil Profile

Top of Layer Depth (ft)	Soil Type	Cohesion (lb/ft2)	N	Angle of Internal Friction (Degrees)	Unit Weight (lb/ft3)
0	Clay	1375	11	0	102
5	Clay	750	6	0	92
7	Clay	750	6	0	92
10	Clay	875	7	0	94
12	Clay	1500	12	0	104
15	Clay	1375	11	0	102
17	Clay	1375	11	0	102
20	Clay	1000	8	0	96
22	Clay	1375	11	0	102
25	Clay	2625	21	0	120
27	Clay	2125	17	0	114
30	Clay	1750	14	0	108
32	Clay	1750	14	0	108
35	Clay	1500	12	0	104
37	Clay	1625	13	0	106
40	Clay	1500	12	0	104
42	Clay	1375	11	0	102
45	Clay	2125	17	0	114
47	Clay	2500	20	0	120
50	Clay	6125	49	0	138

HeliCAP® Summary Report for Upper Guywires
Figure 8-20

HeliCAP SUMMARY REPORT

Job Name: Tower Guy Calculations

Job Number: Lower Guy

Boring No: 1

Anchor Use: Tension

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6/1/2006 8:31:40 AM
Water Table Depth: 22 ft

Capacity Summary

Anchor Number	Anchor Family	Helix Depth (ft)	Helix Capacity (kips)	Total Anchor Capacity (kips)	Recommended Ultimate Capacity (kips)	Torque (ft-lbs)
Anchor 1	Angle: 39 Datum Depth: 0 Length: 25					
14" helix	SS 5	10.6	7.4t 10.2c			
12" helix	SS 5	12.5	7.5t 10.3c			
10" helix	SS 5	14.1	7.1t 6.9c	26.6t	26.6t	3002
8" helix	SS 5	15.4	4.4t 4.2c	31.7c	31.7c	

Soil Profile

Top of Layer Depth (ft)	Soil Type	Cohesion (lb/ft2)	N	Angle of Internal Friction (Degrees)	Unit Weight (lb/ft3)
0	Clay	1375	11	0	102
5	Clay	750	6	0	92
7	Clay	750	6	0	92
10	Clay	875	7	0	94
12	Clay	1500	12	0	104
15	Clay	1375	11	0	102
17	Clay	1375	11	0	102
20	Clay	1000	8	0	96
22	Clay	1375	11	0	102
25	Clay	2625	21	0	120
27	Clay	2125	17	0	114
30	Clay	1750	14	0	108
32	Clay	1750	14	0	108
35	Clay	1500	12	0	104
37	Clay	1625	13	0	106
40	Clay	1500	12	0	104
42	Clay	1375	11	0	102
45	Clay	2125	17	0	114
47	Clay	2500	20	0	120
50	Clay	6125	49	0	138

HeliCAP® Summary Report for Lower Guywires
Figure 8-21

HeliCAP SUMMARY REPORT

Job Name: Tower Foundation Calculations

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6/1/2006 9:27:53 AM
Water Table Depth: 22 ft

Job Number: Three Foundations per Tower Base

Boring No: 1

Anchor Use: Compression

Capacity Summary

Anchor Number	Anchor Family	Helix Depth (ft)	Helix Capacity (kips)	Total Anchor Capacity (kips)	Recommended Ultimate Capacity (kips)	Torque (ft-lbs)
Anchor 1	Angle: 80 Datum Depth: 0 Length: 34					
14" helix	SS 175	25.6	16.9t 23.2c			
12" helix	SS 175	28.5	15.8t 13.8c			
10" helix	SS 175	31	8.9t 8.3c	47.1t	47.1t	5323
8" helix	SS 175	32.9	5.3t 5.3c	50.7c	50.7c	

Soil Profile

Top of Layer Depth (ft)	Soil Type	Cohesion (lb/ft2)	N	Angle of Internal Friction (Degrees)	Unit Weight (lb/ft3)
0	Clay	1375	11	0	102
5	Clay	750	6	0	92
7	Clay	750	6	0	92
10	Clay	875	7	0	94
12	Clay	1500	12	0	104
15	Clay	1375	11	0	102
17	Clay	1375	11	0	102
20	Clay	1000	8	0	96
22	Clay	1375	11	0	102
25	Clay	2625	21	0	120
27	Clay	2125	17	0	114
30	Clay	1750	14	0	108
32	Clay	1750	14	0	108
35	Clay	1500	12	0	104
37	Clay	1625	13	0	106
40	Clay	1500	12	0	104
42	Clay	1375	11	0	102
45	Clay	2125	17	0	114
47	Clay	2500	20	0	120
50	Clay	6125	49	0	138

HeliCAP® Summary Report for Foundations
Figure 8-22

DESIGN EXAMPLES

HUBBELL

DESIGN EXAMPLES

DESIGN EXAMPLE 12
HELICAL ANCHORS for PIPELINE BUOYANCY CONTROL
SYMBOLS USED IN THIS DESIGN EXAMPLE

OD	Outside Diameter	8-53
T_w	Pipe Wall Thickness	8-53
F_y	Minimum Yield Strength of Pipe	8-53
P_d	Pipe Design Pressure	8-53
P_m	Pipe Maximum Operating Pressure	8-53
T_m	Pipe Maximum Operating Temperature	8-53
F	Construction Type Design Factor	8-53
E	Longitudinal Joint Factor	8-53
T	Temperature Factor	8-53
D_c	Density of Coating	8-53
T_c	Thickness of Coating	8-53
D_b	Density of Backfill	8-53
FS	Factor of Safety	8-53
W_p	Weight of Pipe per Linear Foot	8-54
I	Moment of Inertia	8-54
S	Section Modulus	8-54
W_c	Weight of Coating per Linear Foot	8-55
W_g	Gross Buoyancy	8-55
W_n	Net Buoyancy	8-55
L_b	Allowable Span Length Based on Bending Stress	8-55
P	Maximum Design Pressure	8-55
F_h	Hoop Stress	8-55
F_l	Longitudinal Stress	8-55
F_b	Allowable Longitudinal Bending Stress	8-56
M_{max}	Maximum Moment at Mid-Span Between Pipeline Anchor Sets	8-56
L_d	Mid-Span Vertical Displacement Based on Mid-Span Deflection	8-56
Y	Mid-Span Vertical Displacement	8-56
L_p	Allowable Span Length Based on Mechanical Strength of Pipeline Bracket	8-56
UC_p	Ultimate Mechanical Strength of Pipeline Bracket	8-56
WC_p	Working Capacity of Pipeline Bracket	8-56
L_a	Allowable Span Length Based on Uplift Capacity of Anchors in Boring	8-56

UC_a	Ultimate Uplift Capacity	8-56
WC_a	Working Uplift Capacity	8-57
WC_s	Total Working Uplift Capacity	8-57

PURPOSE

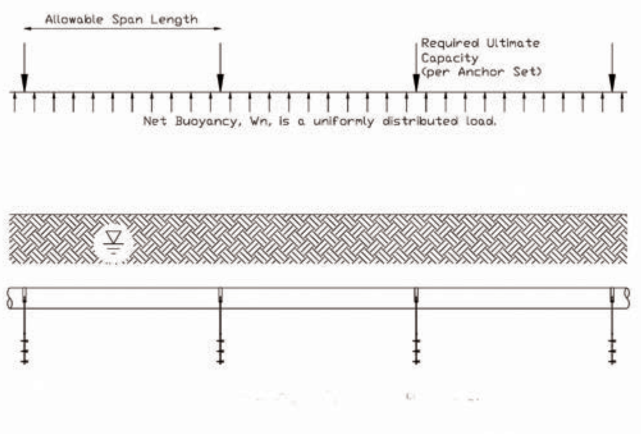
This Design Example provides an aid in the selection of appropriate helical anchors for pipeline buoyancy control.

ASSUMPTIONS

- Pipe contents: Natural gas
- Pipe Outside Diameter (OD): 42"
- Pipe Wall Thickness (T_W): 0.938"
- Grade of Pipe: API 5L, Grade X65
- Minimum Yield Strength Of Pipe (F_y): 65,000 psi
- Pipe design pressure (P_d): 1,440 psi
- Maximum Operating Pressure (P_m): 1,440 psi
- Maximum Operating Temperature (T_m): 85° F
- Construction type design factor (F): 0.50
- Longitudinal joint factor (E): 1.0
- Temperature Factor (T): T_m < 250°F
- Coating: Fusion Bonded Epoxy
- Density of coating (D_c): 70.0 pcf
- Coating thickness (T_c): 16 mils
- Pipeline placement: Land Based in Trench with 4'-0 of Cover above Top of Pipe
- Backfill material: Loose, Poorly Graded Silty Sand
- Specific Gravity of Backfill Material: 1.44
- Density of backfill material (D_b) = 1.44 x 62.4 pcf = 89.9 pcf (use 90.0 pcf)
- Span between anchor sets: Simple Span with Pin-Pin Ends
- Maximum vertical displacement at Mid-Span between Anchor Sets = L_g/360
- Minimum Factor of Safety (FS) for Mechanical Strength Of Hardware/Anchors = 2.0
- Minimum Factor of Safety (FS) for Anchor Soil Capacity = 2.0
- Soil data: As shown in Figure 8-23

Sample Problem - Natural Gas Pipeline			
Borehole BH-1			
HeliCAP® Software Input Values			
Depth (ft)	Clay Cohesion (psf)	Sand N-Value (SPT)	Soil
0		7	sand
3		7	sand
5		28	sand
7		21	sand
10		30	sand
12		21	sand
13	60		clay
15	60		clay
20	380		clay
25	500		clay
30	250		clay
35	460		clay
40	1250		clay
45	2000		clay
50	1560		clay
55	1250		clay
60	2250		clay
65	1320		clay
70	750		clay
75	750		clay

Borehole BH-1 Sample Data
Figure 8-23



Schematic Diagram
Figure 8-24

DESIGN EXAMPLES

SOLUTION

Net Buoyancy (W_n)

Properties of pipe:

- Weight per linear foot (W_p):

$$W_p = [D_s \times \pi \times (42.0^2 - 40.124^2)] / (4 \times 144) \quad \textbf{Equation 8-60}$$

$$= [490.0 \times \pi \times (1764.0 - 1609.935)] / (576)$$

$$= 411.74 \text{ plf}$$

- Moment of inertia (I) = 25515.8 in^4
- Section modulus (S) = 0.7032 ft^3

CHANCE ATLAS

Properties of coating:

- Weight per linear foot (W_c):

W_c = $[D_c \times \pi \times (42.032^2 - 42.0^2)] / (4 \times 144)$ **Equation 8-61**

= $[70.0 \times \pi \times (42.032^2 - 42.0^2)] / (4 \times 144)$

= 1.03 plf

Buoyancy:

- Gross buoyancy (W_g):

W_g = $[D_b \times \pi \times (42.032^2/12^2)] / 4$ **Equation 8-62**

= $[90.0 \times \pi \times (42.032^2/12^2 / 4$

= 865.8 plf

- Net buoyancy (W_n):

W_n = $W_g - W_p - W_c$ **Equation 8-63**

= 865.8 - 411.74 - 1.03

= 453.03 plf (use 453.0 plf)

Allowable Span Length (L_b) Based on Bending Stress

- Maximum design pressure (P):

P = $[(2 \times f_y \times T_w)/OD] \times F \times E \times T$ **Equation 8-64**

= [(2 x 65,000 x 0.938)/42.0] x 0.5 x 1.0 x 1.0

= 1451.7 psi (use given P_d of 1440.0 psi)

- Hoop stress (F_h):

F_h = $(P_d \times OD)/(2 \times T_w)$ **Equation 8-65**

= (1440.0 x 42.0)/(2 x 0.938)

= 32,238.8 psi

- Longitudinal stress (F_l):

F_l = $(0.25 \times P_d \times OD)/T_w$ **Equation 8-66**

= (0.25 x 1440.0 x 42.0)/0.938

= 16,119.4 psi

- Allowable longitudinal bending stress (F_b):

$F_b + F_l$ = $0.75 \times (F \times E \times T) \times F_y$ **Equation 8-67**

F_b = [0.75 x (0.5 x 1.0 x 1.0) x 65,000] - 16,119.4

= 8,255.6 psi

DESIGN EXAMPLES

HUBBELL

$F_b = M_{max}/S$ **Equation 8-68**

M_{max} = Maximum moment at mid-span between pipeline anchor sets

where: $= (W_n \times L_b^2)/8$

$L_b = [(8 \times S \times Fb)/W_n]^{1/2}$

$= [(8 \times 0.7032 \times 8255.6 \times 144)/453.0]^{1/2}$

$= 121.5$ ft

Allowable Span Length (L_d) Based on Mid-Span Deflection

- Mid-span vertical displacement (Y) at center of span:

$Y = L_d/360$ **Equation 8-69**

$L_d/360 = (5 \times W_n \times L_d^4) / (384 \times E \times I)$

$L_d = [(384 \times E \times I) / (360 \times 5 \times W_n)]^{1/3}$

$L_d = [(384 \times 29{,}000{,}000 \times 25525.8/144) / (360 \times 5 \times 453.0)]^{1/3}$

$L_d = 134.2$ ft

$Y = (134.2/360) \times 12 = 4.5$ in

Allowable Span Length (L_p) Based on the Mechanical Strength of Pipeline Bracket

- Rated ultimate mechanical strength (UC_p) of pipeline bracket = 80,000 lbs
- Rated mechanical working capacity (WC_p) of pipeline bracket (using FSm of 2.0):

$WC_p = UC_p/FS_m$ **Equation 8-70**

$= 80{,}000/2$

$= 40{,}000$ lbs

$WC_p = (W_n \times L_p/2) \times 2$ **Equation 8-71**

$L_p = WC_p/W_n$

$= 40{,}000/453.0$

$= 88.3$ ft

Allowable Span Length (L_a) Based on the Uplift Capacity of Anchors in Soil (Boring B-1)

- Ultimate uplift capacity (UC_a) ranges from 45,900 to 41,700 lbs with overall anchor depths below ground line of 51'-0 to 60'-0. See Figure 8-25. Use UC_a = 40,000 lbs.
- Working uplift capacity (WC_a) (using FS_s of 2.0):

$WCa = UC_a/FS_s$ **Equation 8-72**

$= 40{,}000/2$

$= 20{,}000$ lbs

- There are two anchors located at each anchor support location along the pipeline, therefore, the total working uplift capacity (WC_s) per anchor set = WC_a x 2 anchors = 20,000 x 2 = 40,000 lbs.

$L_a = WC_s/W_n$ **Equation 8-73**

$= 40{,}000/453.0$

$= 88.3$ ft

SUMMARY

The uplift capacity plot data was obtained from the soil strength parameters shown in Figure 8-23 and capacities generated by HeliCAP® Engineering Software. The maximum span length between anchor sets is limited to 88 ft based on the ultimate mechanical strength of the pipeline brackets and the ultimate uplift capacity of the anchors in the soil boring shown in Figure 8-25.

Only one soil boring was provided along this proposed section of pipeline. If the soil conditions vary at the anchor set locations and the required average installation torque of 4,000 ft-lbs for a span length of 88 ft cannot be achieved at reasonable anchor depths, the span lengths should be reduced as shown in Table 8-8.

Hubbell Power Systems, Inc. manufactures two band types for use with pipeline buoyancy control systems. See Figure 8-26. Each system has advantages depending on the application and local acceptance. Both systems will provide adequate buoyancy control with industry accepted Factors of Safety.

Summary of Design Criteria, Table 8-7

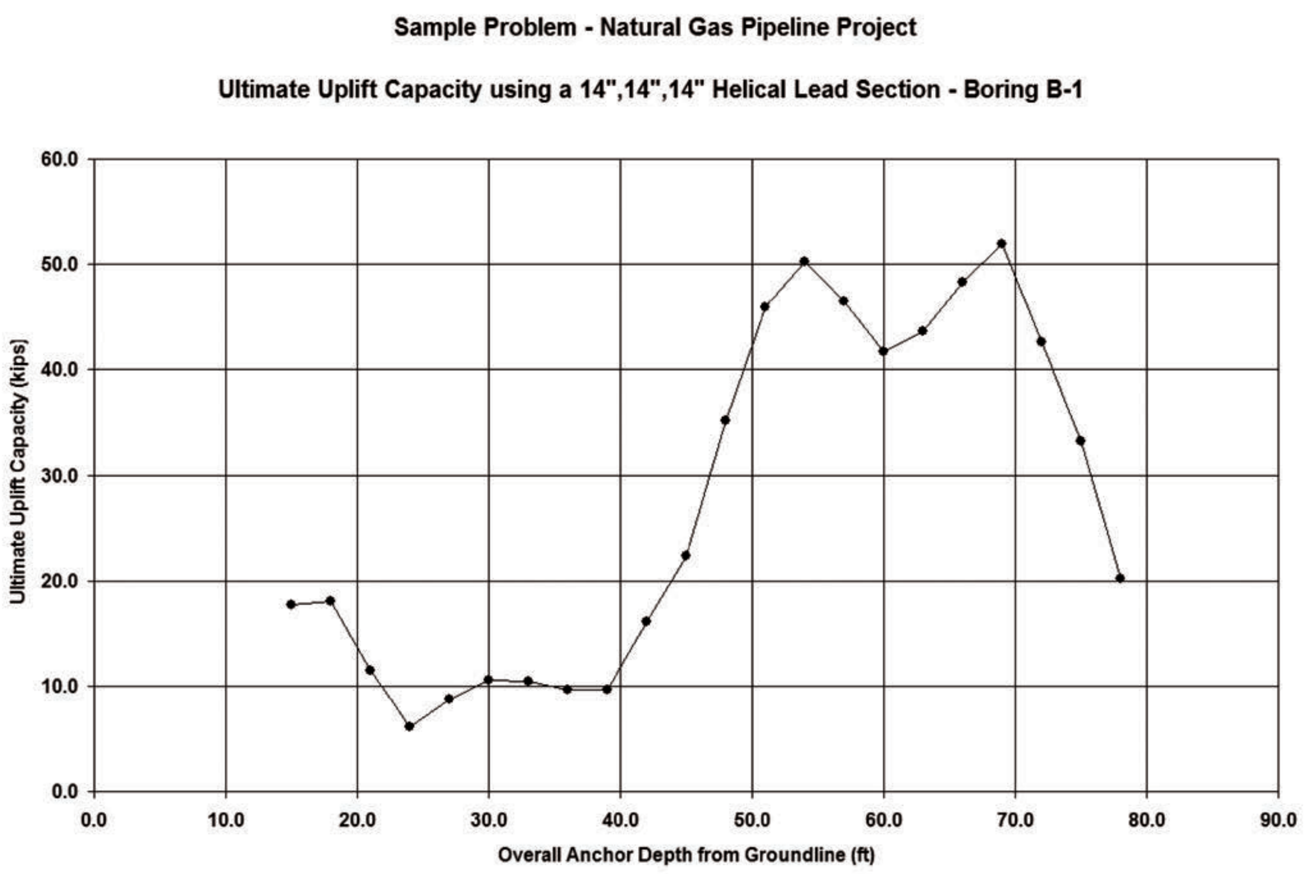
	MAXIMUM ALLOWABLE SPAN LENGTH (ft)	REQUIRED UC_s PER ANCHOR SET (lbs)[2]	REQUIRED UC_a PER ANCHOR SET (lbs)[2]	MINIMUM INSTALLATION TORQUE (ft-lbs)[1,2]
Longitudinal Bending	121.5	110,080	55,040	5,500
Mid-Span Deflection	134.2	121,585	60,793	6,100
Mechanical Strength of Bracket	88.3	80,000	40,000	4,000
Anchor Capacity	88.3	80,000	40,000	4,000

Notes:
1. The required average minimum installation torque is based on using the published installation torque to ultimate capacity ratio (K_t) of 10:1 for the Type SS series anchor material. Torque = UC_a/K_t.
2. These values include a minimum acceptable industry standard Factor of Safety of 2 for helical anchors/piles when used in permanent applications. These pipeline anchors are considered by Hubbell Power Systems, Inc. to be a permanent application. If the client or their representative opts to use a lower Factor of Safety these values will have to be reduced accordingly. For example, at a span length of 88.3 ft, the working capacity per anchor set is 453.0 plf x 88.3 ft = 40,000 lbs. Applying an FS of only 1.5, the required UC_s is 1.5 x 40,000 = 60,000 lbs. The required UC_a is 60,000 lbs/2 anchors = 30,000 lbs. The required minimum installation torque is 30,000/10 = 3,000 ft-lbs.

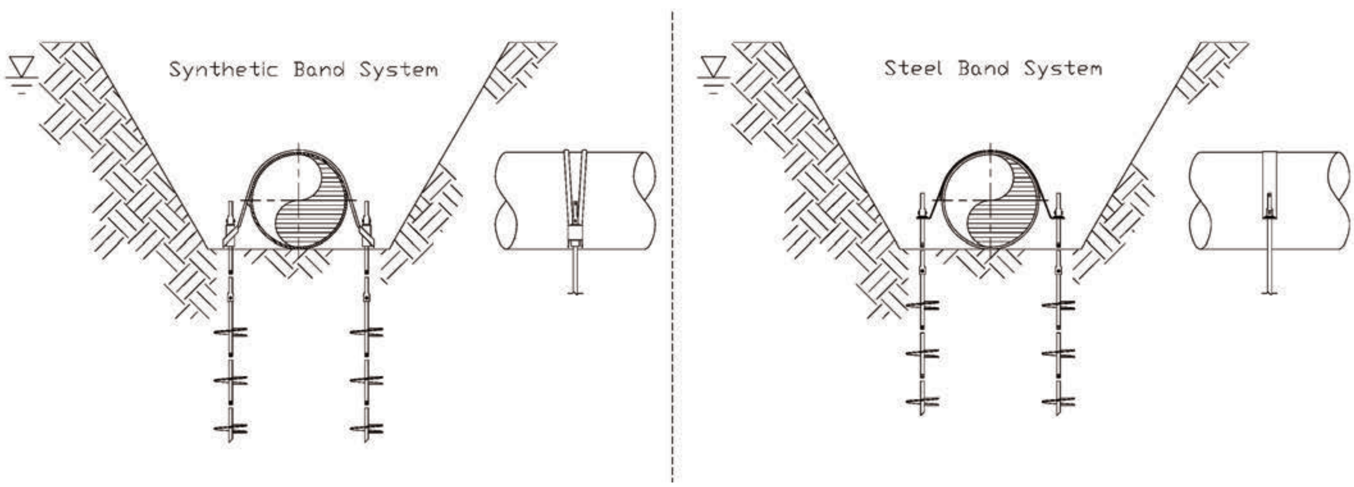
Span Reduction Schedule, Table 8-8

SPAN LENGTH (ft)	REQUIRED UC_s PER ANCHOR SET (lbs)	REQUIRED UC_a PER ANCHOR (lbs)	MINIMUM INSTALLATION TORQUE (ft-lbs)
88	80,000	40,000	4,000
77	70,000	35,000	3,500
66	60,000	30,000	3,000
55	50,000	25,000	2,500
44	40,000	20,000	2,000

DESIGN EXAMPLES



Ultimate Uplift Capacity
Figure 8-25



Band Systems
Figure 8-26

DESIGN EXAMPLE 13

TYPE RS HELICAL PILES for LATERAL SUPPORT

SYMBOLS USED IN THIS DESIGN EXAMPLE

c	Cohesion Factor of Soil	8-59
P	Applied Horizontal Shear Load	8-59
C_u	Cohesion of Clay	8-59
D	Diameter of Foundation	8-59
e	Eccentricity	8-59
L	Minimum Length of Foundation	8-59
f	Bending Stress	8-59
$M^{POS}{}_{MAX}$	Maximum Bending Moment	8-60
L	Required Depth into Soil	8-60

PROBLEM

A CHANCE® Helical Type SS175 1-3/4" square shaft helical anchor/pile is proposed for a pedestrian bridge abutment. The top section of the shaft is to be encased in a 6" nominal steel pipe and grout to provide lateral resistance. The top ten feet of the soil profile is medium-stiff clay with a cohesion factor (c) of 1000 psf. Determine what length of 6" diameter steel case is required to resist 4400 lbs of lateral load using the Broms' Method.

Assumptions

- The 1-3/4" square shaft below the 6" cased section provides no lateral resistance.
- The solution method used is shown in Figure 8-27.
- Eccentricity is assumed to be 1 ft.

Solution

P	=	Applied horizontal shear load: Use 4400 lbs. Include a Factor of Safety of 2 in the calculations, thus doubling the horizontal shear load; P = 2 x 4400 = 8800 lbs.
C_u	=	Cohesion of clay: Use C_u = 1000 psf
D	=	Diameter of foundation: Use D = 6.625" (6" nominal pipe size)
e	=	Eccentricity; distance above grade to resolved load: Use e = 1 ft
L	=	Minimum length of foundation based on above criteria.

f = P/9 (C_u) D **Equation 8-74**

= 8800 lbs/9 (1000 psf) (6.625 in/12)

= 1.771 ft

DESIGN EXAMPLES

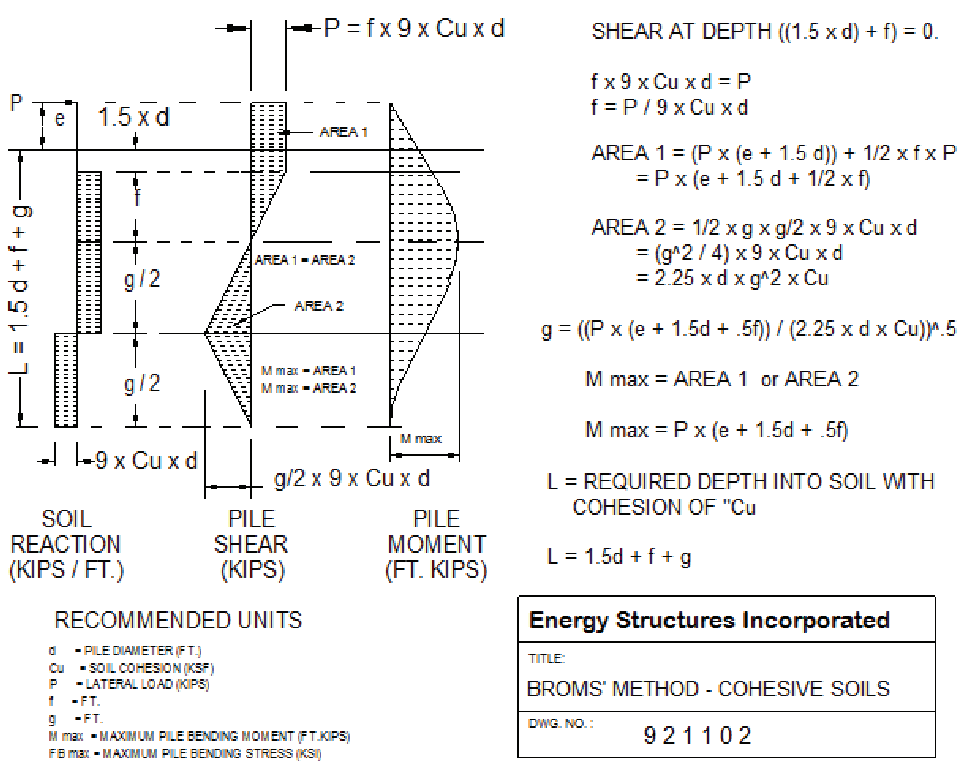
$$M^{POS}{}_{MAX} = P\,[e + 1.5(d) + 0.5(f)] \quad \textbf{Equation 8-75}$$
$$= 8800 \text{ lbs } [1 \text{ ft} + 1.5\,(6.625 \text{ in}/12) + 0.5\,(1.771 \text{ ft})]$$
$$= 23{,}880 \text{ ft-lbs}$$

$$M^{POS}{}_{MAX} = 2.25\,(d)\,g^2\,(Cu) \quad \textbf{Equation 8-76}$$
$$23{,}880 \text{ ft-lbs} = 2.25\,(6.625 \text{ in}/12)\,g^2\,(1000 \text{ psf})$$
$$g^2 = 19.22 \text{ ft}^2$$
$$g = \sqrt{19.22}$$
$$= 4.38 \text{ ft}$$

$$L = 1.5D + f + g \quad \textbf{Equation 8-77}$$
$$= 1.5\,(6.625 \text{ in}/12) + 1.771 \text{ ft} + 4.38 \text{ ft}$$
$$= 6.98 \text{ ft}$$

Summary

The 6” nominal steel case should be at least 7’-0 long to resist the 4400 lb lateral load.



Broms’ Method for Laterally Loaded Short Piles
Figure 8-27

DESIGN EXAMPLE 14
INSTANT FOUNDATIONS® for STREET LIGHT SUPPORTS
SYMBOLS USED IN THIS DESIGN EXAMPLE

SLF	Street Light Foundation	8-61
DL	Dead or Down Load	8-61
V	Horizontal or Lateral Shear Load	8-61
M	Moment Loads	8-61
AASHTO	American Association of State Highway and Transportation Officials	8-61
L	Required Length	8-63
c	Cohesion of Soil	8-63
FS	Factor of Safety	8-63
V_F	Applied Shear at Groundline including Factor of Safety	8-63
V_M	Applied Moment at Groundline including Factor of Safety	8-63
D	Diameter of Foundation	8-63
q	Broms' Coefficient	8-63
M_{MAX}	Maximum Moment Applied to Foundation	8-63
φ	Internal Angle of Friction	8-64
γ	Unit Weight of Soil	8-64
K_p	Passive Earth Pressure Coefficient	8-64

PURPOSE

This Design Example provides example solutions to aid in the selection of appropriate CHANCE® Helical Instant Foundation® products for different job parameters.

SLF LOADS

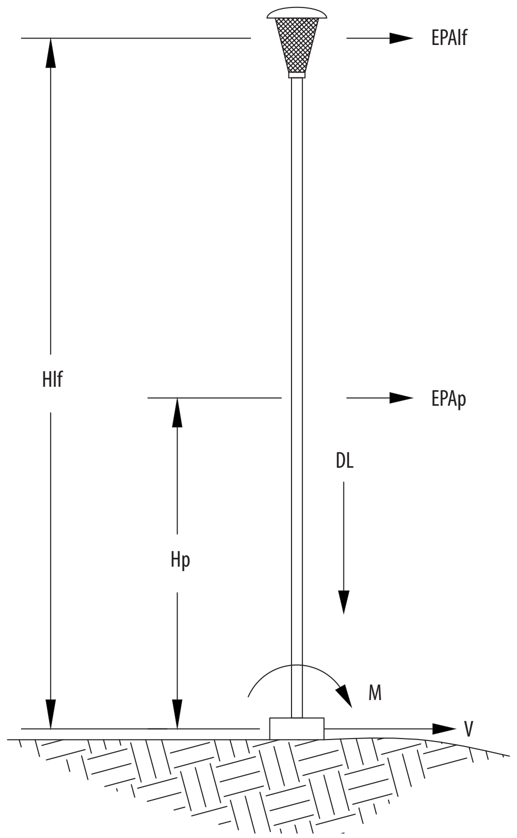
The resulting pole loads to be resisted by a street light foundation (SLF) are dead or vertical down loads (DL), horizontal, lateral or shear loads (V) due to wind on the pole and luminaire (light fixture), and overturning moment loads (M) resulting from the tendency to bend at or near the ground line as the wind causes the pole to displace and the foundation restrains the pole base at one location (see Figure 8-28).

The DL for an SLF application is so small that a foundation sized to resist V and M will typically be much more than adequate to resist DL. Therefore, DL will not control the SLF design and will not be considered here. If DL is large enough to be of concern for an application where an SLF will be used, it may be evaluated based on bearing capacity equations applied to the soil around the helical bearing plate and friction along the shaft. These evaluations are beyond the scope of this design example, which will only deal with SLF applications.

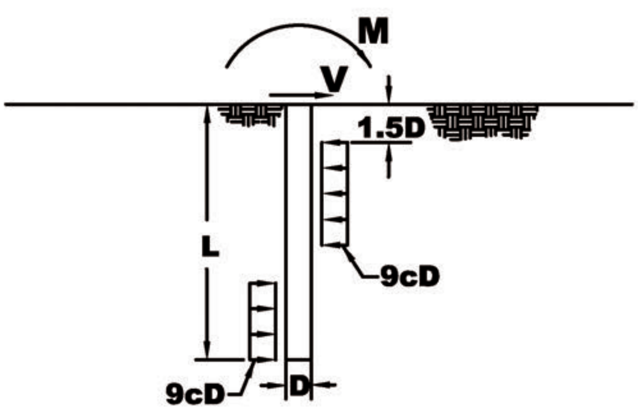
Since SLF products are used as lighting foundations along public highways, it is appropriate to mention the American Association of State Highway and Transportation Officials (AASHTO) publication *Standard Specifications for Structural Support for Highway Signs, Luminaires and Traffic Signals*. This document is often taken as the controlling specification for jobs using SLF's and will be referenced throughout this discussion.

wp = Wind Pressure
EPAlf = Effective Projected Area of a Light Fixture
EPAp = Effective Projected Area of a Light Pole
Hlf = Moment Arm to EPAlf Centroid
Hp= Moment Arm to EPAp Centroid

SLF REACTIONS
Vlf = [EPAlf x wp]
Vp = [EPAp x wp]
V = Vlf +Vp
M = [Vlf x Hlf] + [Vp x Hp]



Pole Load Diagram
Figure 8-28



Foundation in Cohesive Soil
Figure 8-29

SLF SELECTION

The SLF selection process is a trial and error procedure that may require more than one iteration. First, select an SLF diameter based on the applied bending moment (M) that must be resisted. That is, ensure that the applied moment is less that the allowable moment on the shaft. Determining the allowable moment requires a structural analysis of the pipe shaft section capacities (often based on a reduced cross section through cable ways, bolt slots, base plate size, welds, etc). This effort should be familiar to engineers engaged in design work, so a sample of this process will not be given here.

The foundation shaft diameter will often be as large as or larger than the base diameter of the pole to be supported. Allowable moment capacities for CHANCE® Helical Instant Foundation® products are provided in Table 10-2 in Section 10 of this Technical Design Manual. These capacities, when compared to the ground line reactions of the pole, can be used to choose a starting diameter to resist the applied loads. In this regard, shear is usually not the controlling factor for SLF shaft size but rather the moment load. (Note: The starting size may change as the given soil conditions for a job may dictate the final SLF size required.)

The design or selection of a foundation size to resist light pole loads in a given soil may be determined by various methods. Numerical methods using finite element and finite difference techniques may be used but have proven to be somewhat sophisticated for the rather simple SLF application. The Fourth Edition of the AASHTO specification lists a number of preliminary design methods that can be employed in the design process. Among those listed and discussed are the methods developed by Bengt B. Broms for embedment lengths in cohesive and cohesionless soils and a graphical method dealing with the embedment of lightly loaded poles and posts. The Broms method will be used for this design example as experience has shown these methods to both useable and appropriate. Calculations are provided for both cohesive soil (clay) and cohesionless soil (sand).

COHESIVE SOIL (see Figure 8-29)

Assumed values:

- Applied shear load at the groundline (V) = 460 lbs.
- Applied moment at the groundline (M) = 8600 ft-lbs.
- Foundation diameter is 6" nominal Schedule 40. Use 6.625" as the actual pipe size in calculations. Cableway openings are 2.5" wide by 12" high. The allowable moment capacity of this foundation shaft size and cableway opening is 10,860 ft-lbs.
- The required length (L) will be determined using the Broms method.
- Cohesion (c) = 1000 psf.
- Factor of Safety = 2.

V_F = V (FS) **Equation 8-78**

= 460 (2)

= 920 lbs

V_M = M (FS) **Equation 8-79**

= 8600 (2)

= 17,200 ft-lbs

L = 1.5D+q [1+{ 2 + (4H+6D)/q} $^{0.5}$] **Equation 8-80**

= 1.5 (6.625/12) + 0.185157 x [1 + { 2+ (4 x 18.69565 + 6 x (6.625/12)) / (0.185157)} $^{0.5}$]

= 4.82 ft

where:

D = Diameter of foundation = 6.625 inches

q = V_F/9cD = 920 / (9 x 1000 x 6.625/12) = 0.185157ft

c = Shear strength of cohesive soil = 1000 psf

H = Moment / Shear = M/V = V_M / V_F = 17200 ft-lbs / 920 lbs = 18.69565 ft

L = Calculated Foundation Length to Provide a SF of 2 Against Soil Failure.

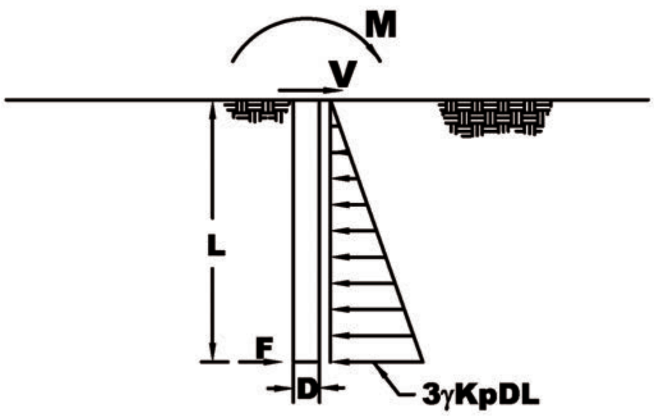
The length required to provide a Factor of Safety of 2 against soil failure is 4.82 ft. Since SLF lengths are provided in even foot lengths, use L = 5 ft. For the required embedment length, the maximum moment in the shaft is:

M_{MAX} = V (H + 1.5D + 0.5q) **Equation 8-81**

= 460 (18.69565 + (1.5 x 6.625/12) + (0.5 x 0.185157)

= 9023.5 ft-lbs

Maximum moment can be compared with the allowable moment capacity of the foundation shaft to determine adequacy. For this example the allowable moment in the 6" pipe shaft is given as 10,860 ft-lbs, which is greater than the applied moment. Therefore, the 6" diameter by 5' long SLF is adequate for the applied loads in the clay soil.



Foundation in Cohesionless Soil
Figure 8-30

COHESIONLESS SOIL (See Figure 8-30)

Assumed values:

- Applied shear load at the groundline (V) = 460 lbs.
- Applied moment at the groundline (M) = 8600 ft-lbs.
- Foundation diameter is 6" nominal Schedule 40. Use 6.625" as the actual pipe size in calculations. Cableway openings are 2.5" wide by 12" high. The allowable moment capacity of this foundation shaft size and cableway opening is 10,860 ft-lbs.
- The required length (L) will be determined using the Broms method.
- $\varphi = 30°$
- $\gamma = 100$ lbs/ft3
- Factor of Safety = 2.

V_F = V (FS) = 460 (2) = 920 lbs **Equation 8-78**

V_M = M (FS) = 8600 (2) = 17,200 ft-lbs **Equation 8-79**

Broms equation for cohesionless soil requires a trial and error solution. For the trial and error solution, start by assuming the foundation diameter (D) is 6.625" and the length (L) is 6 feet:

$$0 \le L^3 - (2V_F L / K_{P\gamma} D) - (2VM / K_{P\gamma} D)$$ **Equation 8-82**

$$= 6^3 - [2 \times 920 \times 6) / (3 \times 100 \{6.625/12\})] - [(2 \times 17200) / (3 \times 100 \times \{6.625/12\})]$$

$$= -58.35$$

$$0 > -58.35$$

where:

$K_P = \tan^2 (45 + \varphi/2) = 3.0$

γ = Effective unit weight of soil = 100 lbs/ft^3

The 6 foot length is too short so we will try a 7 foot length and repeat the calculation:

$$0 = 7^3 - [2 \times 920 \times 7) / (3 \times 100 \{6.625/12\})] - [(2 \times 17200) / (3 \times 100 \times \{6.625/12\})]$$

$$= 57.53$$

$$0 < 57.53$$

A 7 foot long SLF will be adequate. The maximum moment in the foundation shaft can be determined with the following equation:

$$M_{MAX} = V (H + 0.54 \times (V / \gamma D K_P)^{0.5})$$ **Equation 8-83**

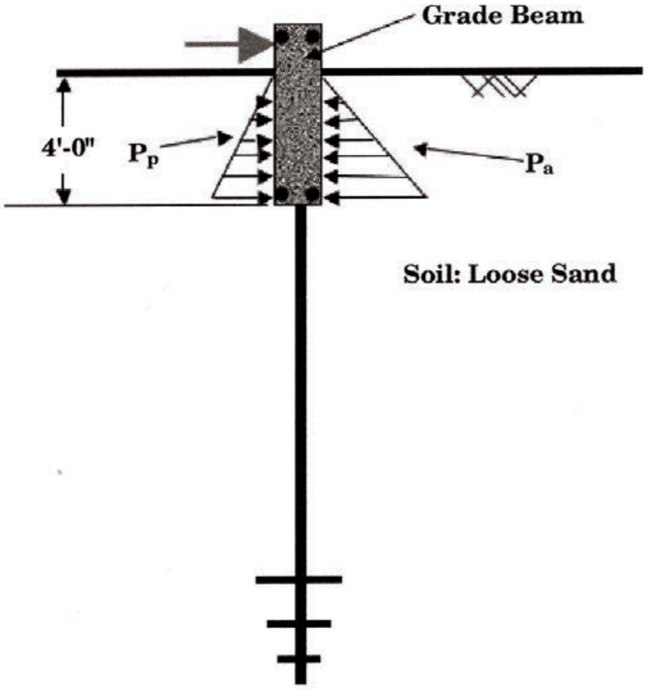
$$= 460 (18.69565 + 0.54 \times (460/100 \times (6.625/12) \times 3)^{0.5})$$

$$= 9013.968 \text{ ft-lbs}$$

This is less than the allowable moment capacity of 10,860 ft-lbs, therefore a 6" diameter by 7' long SLF is adequate for the applied load in the sandy soil.

DESIGN EXAMPLE 15
FOUNDATION EARTH PRESSURE RESISTANCE
SYMBOLS USED IN THIS DESIGN EXAMPLE

pcf .... Pounds per Cubic Foot 8-65
K_a .... Active Earth Pressure Coefficient 8-65
K_p .... Passive Earth Pressure Coefficient 8-65
P_a .... Active Load 8-66
P_p .... Passive Load 8-66



Earth Pressure on a Grade Beam
Figure 8-31

PROJECT

A CHANCE® Helical Type SS5 1-1/2" square shaft helical anchor is proposed as part of a pier and beam foundation for a residential structure (see Figure 8-31). The top of the helical anchor is fixed in a concrete grade beam that extends 4'-0 below grade. The surface soils are loose sands. Determine the lateral capacity of the grade beam using the Rankine earth pressure method.

ASSUMPTIONS

- The lateral capacity of the 1-1/2" square shaft helical anchor is limited based on shaft size. It is generally not assigned any contribution to the lateral capacity of a foundation
- The effective length of the grade beam for lateral resistance is 25'-0
- Assume a unit weight of 95 pcf
- The water table is well below the bottom of the grade beam
- There are no surcharge loads
- From Table 8-9, $K_a = 0.2$, $K_p = 3$

SOLUTION

P_a	=	$0.5K_a\gamma H^2$	**Equation 8-84**
	=	0.5 x 0.2 x 95 x 42	
	=	152 lb/ft	
P_p	=	$0.5K_p\gamma H^2$	
	=	0.5 x 3 x 95 x 42	
	=	2280 lb/ft	
P_p - P_a	=	2280 - 152	
	=	2128 lb/ft	
Total lateral resistance	=	2128 x 25'-0 = 53,200 lbs	

NOTE: In this example, more than 1" of movement will probably be required to fully mobilize the total lateral resistance. Partial mobilization requires less deflection.

Coefficients of Earth Pressure (Das, 1987), Table 8-9

SOIL	K_0' DRAINED	K_0' TOTAL	K_a' TOTAL	K_p' TOTAL
Clay, soft [1]	0.6	1	1	1
Clay, hard [1]	0.5	0.8	1	1
Sand, loose	0.6	0.53	0.2	3
Sand, dense	0.4	0.35	0.3	4.6

Note:
[1] Assume saturated clays.

DESIGN EXAMPLES

DESIGN EXAMPLE 16
BUCKLING EXAMPLE USING the DAVISSON METHOD
SYMBOLS USED IN THIS DESIGN EXAMPLE

k_h	Empirical Torque Factor for Helix	8-67
U_{cr}	Critical Capacity	8-67
R	Resistance	8-68
I_{max}	Maximum Moment of Inertia	8-68
P_{cr}	Critical Pressure	8-68
E_p	Modulus of Elasticity	8-68
I_p	Moment of Inertia	8-68
D	Shaft Diameter	8-68
kip	Kilopound	8-68

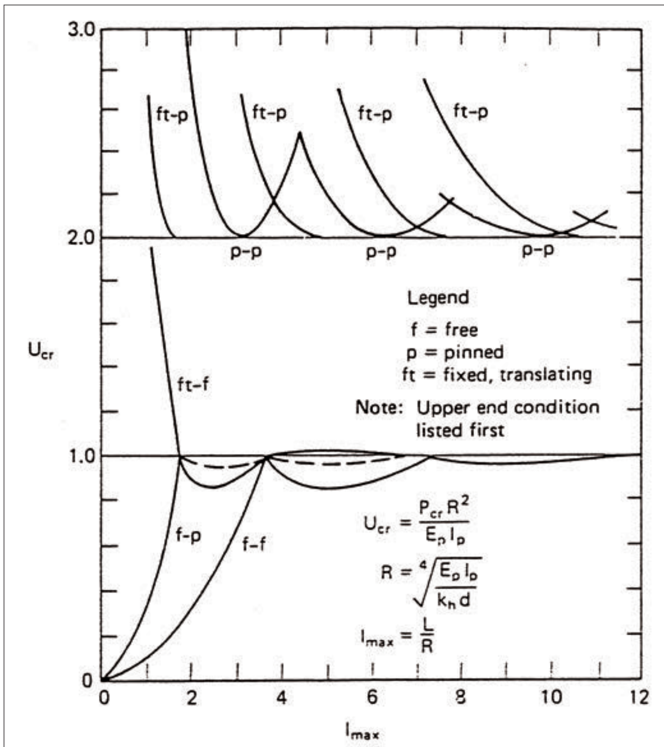
DESIGN EXAMPLES

PROJECT

A three-helix CHANCE® Helical Type SS150 1-1/2" square shaft helical pile is to be installed into the soil profile as shown in Figure 8-33. The top three feet is uncontrolled fill and is assumed to be soft clay. The majority of the shaft length (12 feet) is confined by soft clay with a k_h = 15 pci. The helix plates will be located in stiff clay below 15 feet. The buckling model assumes a pinned-pinned end condition for the helical pile head and tip. Determine the critical buckling load using the Davisson method.

ASSUMPTIONS

- k_h is constant, i.e., it does not vary with depth. This is a conservative assumption because k_h usually varies with depth, and in most cases increases with depth.
- Pinned-pinned end conditions are assumed. In reality, end conditions are more nearly fixed than pinned, thus the results are generally conservative.
- From Figure 8-32, $U_{cr} \approx 2$

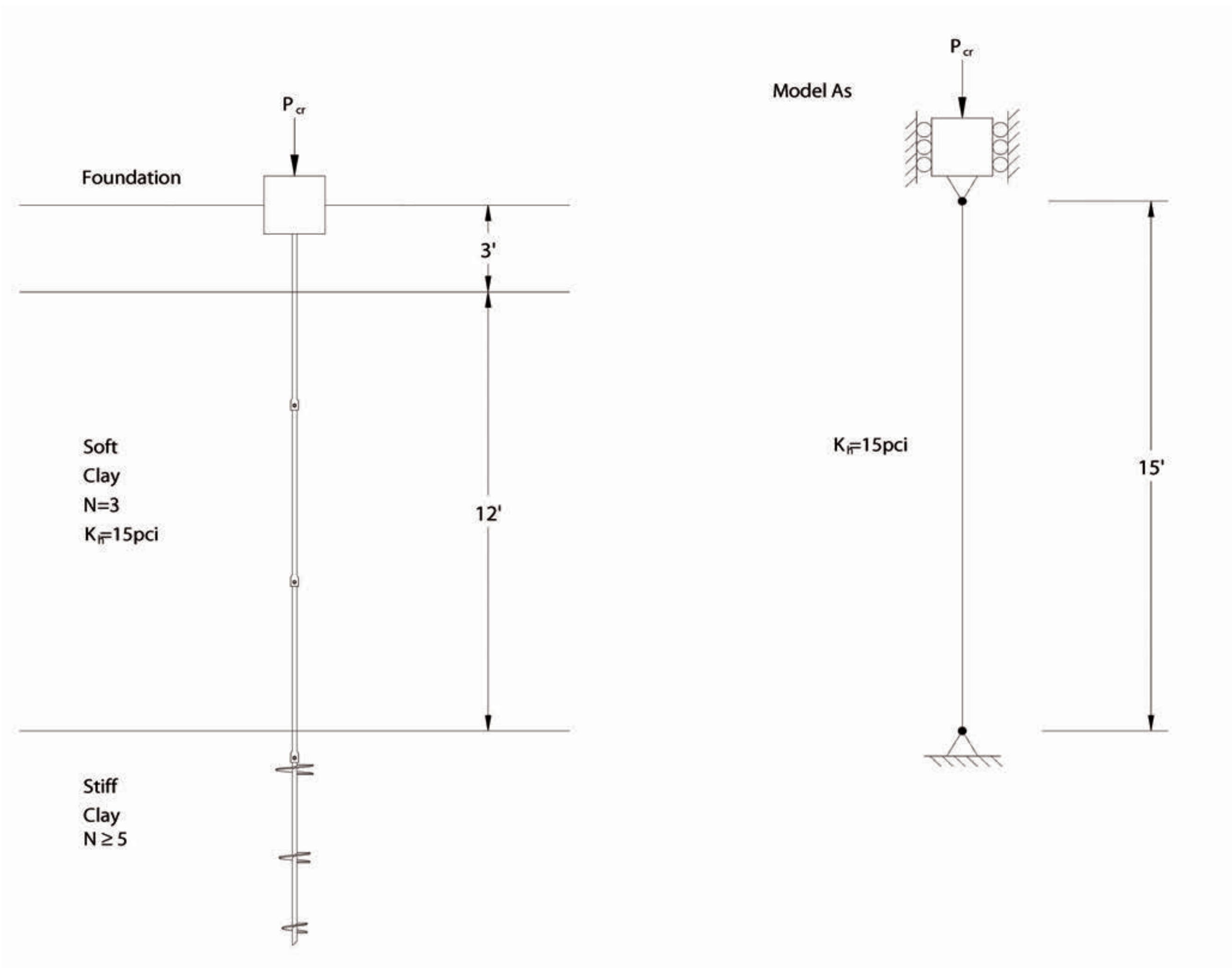


Poulos and Davis (1980)
Figure 8-32

R	=	$\sqrt[4]{(30 \times 10^6 \times 0.396) / (15 \times 1.5)} = 26.96$	**Equation 8-85**
I_{max}	=	$(15 \times 12) / 26.96$	
	=	6.7	
P_{cr}	=	$(2 \times 30 \times 106 \times 0.396) / 26.96^2$	
	=	32.69 kips	

CHANCE® Helical Type SS150 Square Shaft Foundations Physical Properties, Table 8-10

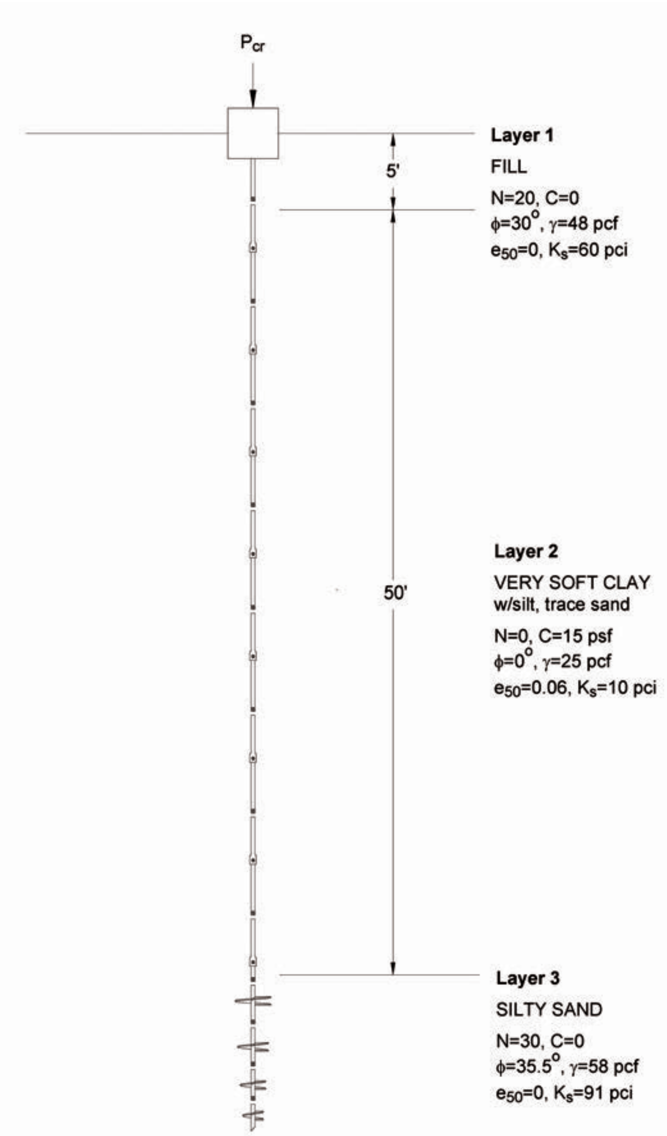
MODULUS of ELASTICITY (E_p)	MOMENT of INERTIA (I_p)	SHAFT DIAMETER (D)
30×10^6 psi	0.396 in^4	1.5 in



Foundation Details
Figure 8-33

DESIGN EXAMPLE 17
BUCKLING EXAMPLE USING the FINITE DIFFERENCE METHOD
SYMBOLS USED IN THIS DESIGN EXAMPLE

WOH	Weight of Hammer	8-69
WOR	Weight of Rod	8-69
psf	Pounds per Square Foot	8-70
ID	Inside Diameter	8-70
HPM	CHANCE HELICAL PULLDOWN® Micropile	8-70



Foundation Details
Figure 8-34

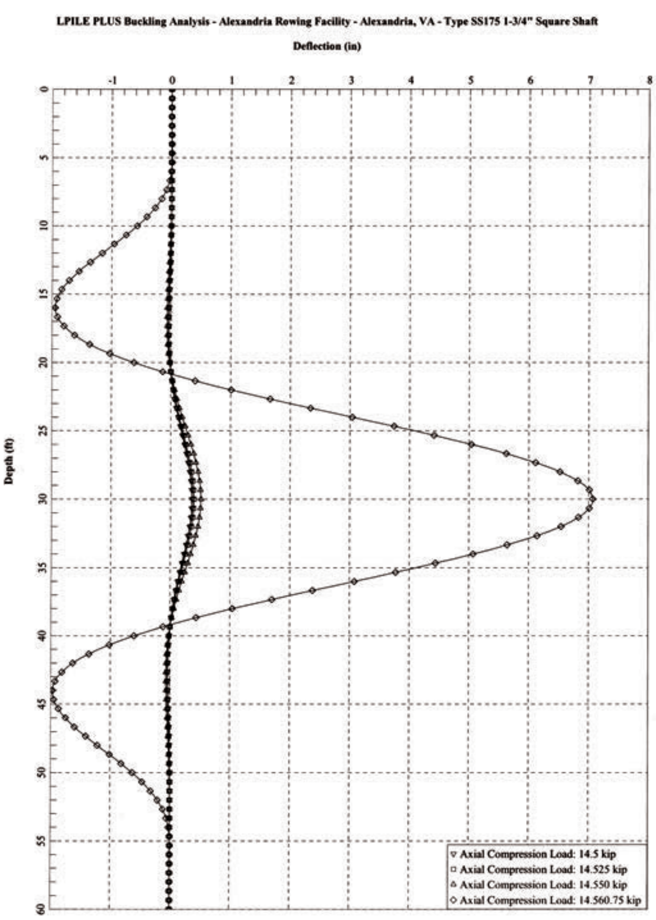
A four-helix CHANCE® Helical Pile is to be installed into the soil profile as shown in Figure 8-34. The top five feet is compacted granular fill and is considered adequate to support lightly loaded slabs and shallow foundations. The majority of the shaft length (50 feet) is confined by very soft clay described by the borings as “weight of hammer” (WOH) or “weight of rod” (WOR) material. WOH or WOR material means the weight of the 130-lb drop hammer or the weight of the drill rod used to extend the sampler down the borehole during the standard penetration test is enough to push the sampler down 18+ inches. As a result, a low cohesion value (15 psf) is assumed. The helix plates will be located in dense sand below 55 feet. Determine the critical buckling load of a Type SS175 1-3/4” square shaft and Type RS3500.300 round shaft piles using LPILEPLUS 3.0 for Windows® (ENSOFT, Austin, TX).

When the computer model is completed, the solution becomes an iterative process of applying successively increasing loads until a physically logical solution converges. At or near the critical buckling load, very small increasing increments of axial load will result in significant changes in lateral deflection – which is a good indication of elastic buckling. Figure 8-35 is an LPILEPLUS output plot of lateral shaft deflection vs depth. As can be seen by the plot, an axial load of 14,561 lb is the critical buckling load for a Type SS175 1-3/4” square shaft because of the dramatic increase in lateral deflection at that load compared to previous lesser loads. Figure 8-36 indicates a critical buckling load of 69,492 lb for Type RS3500.300 round shaft.

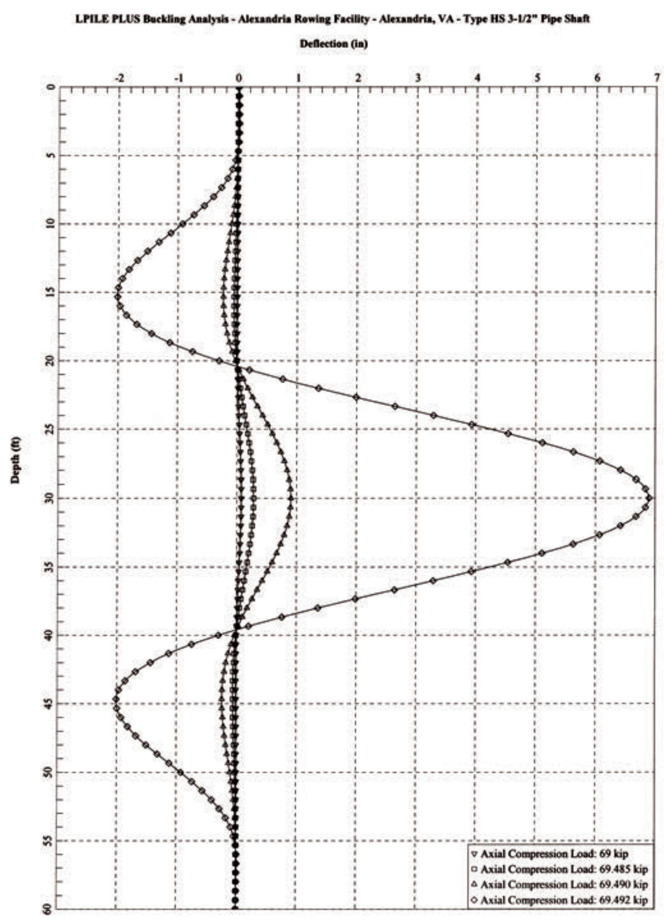
Note that over the same 50-foot length of very soft clay, the well-known Euler equation predicts a critical buckling load for Type SS175 of 614 lb with pinned-pinned end conditions and 2,454 lb with fixed-fixed end conditions. The Euler critical buckling load for Type RS3500.300 is 3,200 lb for pinned-pinned and 12,800 lb for fixed-fixed. This is a good indication that shaft confinement provided by the soil will significantly increase the buckling load of helical piles. This also indicates that even the softest materials will provide significant resistance to buckling.

All extendable helical piles have couplings or joints used to connect succeeding sections together in order to install the helix plates in bearing soil. One inherent disadvantage of using the finite difference method is its inability to model the effects of bolted couplings or joints that have zero joint stiffness until the coupling rotates enough to bring the shaft sides into contact with the coupling walls. This is analogous to saying the coupling or joint acts as a pin connection until it has rotated a specific amount, after which it acts as a rigid element with some flexural stiffness. All bolted couplings or joints, including square shaft and round shaft piles, have a certain amount of rotational tolerance. This means the joint initially has no stiffness until it has rotated enough to act as a rigid element. In these cases, it is probably better to conduct buckling analysis using other means, such as finite element analysis, or other methods based on empirical experience as mentioned earlier.

If couplings are completely rigid, i.e., exhibit some flexural stiffness even at zero joint rotation, axial load is transferred without the effects of a pin connection, and the finite difference method can be used. An easy way to accomplish rigid couplings with round shaft piles is to pour concrete or grout down the ID of the pipe after installation. Another method is to install a grout column around the square or round shaft of the foundation using the CHANCE HELICAL PULLDOWN® Micropile (HPM) method. The HPM is a patented (U.S. Patent 5,707,180) installation method initially developed to install helical anchor foundations in very weak soils where buckling may be anticipated.



LPILE^{PLUS} Output Plot of Deflection vs Depth
Figure 8-35

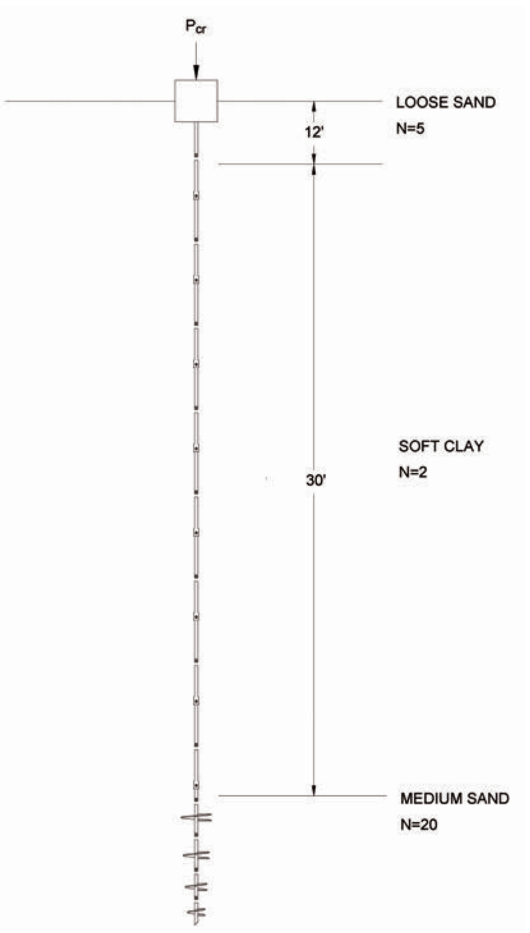


LPILE^{PLUS} Output Plot of Deflection vs Depth
Figure 8-36

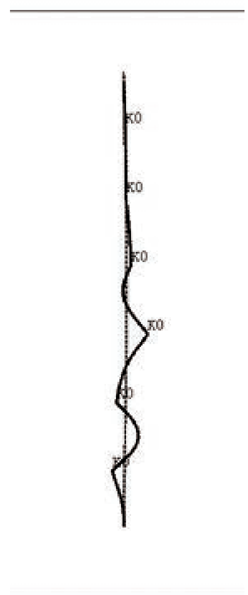
DESIGN EXAMPLE 18
BUCKLING EXAMPLE USING the FINITE DIFFERENCE METHOD

SYMBOLS USED IN THIS DESIGN EXAMPLE

SPT .... Standard Penetration Test 8-71
N .... SPT Blow Count 8-71
psf .... Pounds per Square Foot 8-71
kip .... Kilopound 8-71
HPM .... CHANCE HELICAL PULLDOWN® Micropile 8-71



Foundation Details
Figure 8-37



Displaced Shape of Shaft ANSYS® Output
Figure 8-38

A three-helix CHANCE® Helical Type SS5 1-1/2" square shaft helical pile is to be used to underpin an existing townhouse structure that has experienced settlement (see Figure 8-37 for soil profile details). The top 12 feet is loose sand fill, which probably contributed to the settlement problem. The majority of the shaft length (30 feet) is confined by very soft clay with an SPT blow count "N" of 2. As a result, a cohesion value (250 psf) is assumed. The helix plates will be located in medium-dense sand below 42 feet. Determine the critical buckling load using the ANSYS integrated file element model.

Output indicates the Type SS5 1-1/2" square shaft buckled at around 28 kip. Figure 8-38 shows the displaced shape of the shaft (exaggerated for clarity). The "K0" in Figure 8-38 are the locations of the shaft couplings. Note that the deflection response is controlled by the couplings, as would be expected. Also note that the shaft deflection occurs in the very soft clay above the medium-dense bearing stratum. Since the 28 kip buckling load is considerably less than the bearing capacity (55+ kip) it is recommended to install a grout column around the 1-1/2" square shaft using the CHANCE HELICAL PULLDOWN® Micropile (HPM) method.